William P. Morse

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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(INSTITUTED 1852)

VOL. XL—No. 1

JANUARY, 1914

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1914

American Society of Civil Engineers

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ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard, George W. Tillson, Nelson P. Lewis, Charles J. Tilden.

ON VALUATION OF PUBLIC UTILITIES: Frederic P. Stearns, H. M. Byllesby, Thomas H. Johnson, Leonard Metcalf, Alfred Noble, William G. Raymond, Jonathan P. Snow.

TO INVESTIGATE CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS: Alfred Noble, S. L. F. Deyo, Dugald C. Jackson, William V. Judson, George W. Tillson, C. F. Loweth, John A. Bensel.

TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edward C. Shankland, Edwin Duryea, Jr., James C. Meem, Walter J. Douglas, Samuel T. Wagner, Frank M. Kerr.

ON A NATIONAL WATER LAW: F. H. Newell, George G. Anderson, Charles W. Comstock, Clemens Herschel, W. C. Hoad, Robert E. Horton, John H. Lewis, Charles D. Marx, Gardner S. Williams.

ON FLOODS AND FLOOD PREVENTION: Frank M. Kerr, John A. Bensel, T. G. Dabney, C. E. Grunsky, Morris Knowles, J. B. Lippincott, Daniel W. Mead, John A. Ockerson, Arthur T. Safford, Charles Saville, F. L. Sellew, C. McD. Townsend.

TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Emil Gerber, Robert W. Hunt, George W. Kittredge, William McNab, G. J. Ray, F. E. Turneaure, J. E. Willoughby.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER....................5913 Columbus.
CABLE ADDRESS...................."Ceas, New York."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

December 17th, 1913.—The meeting was called to order at 8.30 P. M.; Vice-President J. Waldo Smith in the chair; Chas. Warren Hunt, Secretary; and present, also, 197 members and 33 guests.

A paper by John W. Alvord, M. Am. Soc. C. E., entitled "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return," was presented in abstract by the Secretary, who also read a communication on the subject from W. J. Wilgus, M. Am. Soc. C. E. Owing to lack of time, discussions by Messrs. J. E. Willoughby, Frank C. Boes, Allen Hazen, H. C. Vensano, and Leonard Metcalf, were read by title only. The paper was discussed orally by Messrs. Alexander C. Humphreys, F. Lavis, Henry Floy, and Charles Rufus Harte.

A paper by Allen Hazen, M. Am. Soc. C. E., entitled "Storage to be Provided in Impounding Reservoirs for Municipal Water Sup-

Manley Osgood, Ann Arbor, Mich.
Harold Crosby Stevens, Charlotte, N. C.
Marion Ross Watson, Boca Grande, Fla.
Chauncey Earl Webb, Fort Shaw, Mont.
Liang Yu, Chicago, Ill.

The Secretary announced the transfer of the following candidates on December 31st, 1913:

From Associate Member to Member

Harold Sherburne Boardman, Orono, Me.
Frank Colburn Bowler, Millinocket, Me.
Albert Valdemar Gude, Jr., Atlanta, Ga.
Linn Murdoch Huntington, New York City
John Robert Clarke Macredie, Prussia, Saskatchewan, Canada
William Clare Spiker, Atlanta, Ga.
Fred Caswell Stanton, Cristobal, Canal Zone, Panama

From Associate to Associate Member

David Walker Brooks, New York City

From Junior to Associate Member

Paul Jones Bean, Pearl Harbor, Hawaii
Charles Smith Bilyeu, New York City
Leon George Cutler, New York City
Arthur William Harrington, Potsdam, N. Y.
Theodore Sedgwick Johnson, Granville, Ohio
Frederic William Lyon, Pittsburgh, Pa.
Charles Reed Marsh, Johnstown, Pa.
James Elwood Payne, Youngstown, Ohio
Hubert Earl Snyder, Logan, West Va.
Albert Irvine Stiles, Barrios, Guatemala

The Secretary announced the following deaths:

Robert Cooke Clarkson, of Philadelphia, Pa., elected Junior, January 5th, 1887; Member, January 2d, 1901; died December 26th, 1913.

Joseph P. Cotton, of Newport, R. I., elected Member, June 7th, 1876; died December 13th, 1913.

Henry Francis Labelle, of Albuquerque, N. Mex., elected Member, April 6th, 1898; died December 12th, 1913.

John Wiley Miles, of City of Mexico, Mexico, elected Associate Member, October 2d, 1901; Member, April 6th, 1909; died in January, 1912.

Murray Forbes, of Greensburg, Pa., elected Associate Member, June 5th, 1907; died December 28th, 1913.

Harry Woy Gray, of Los Angeles, Cal., elected Associate Member, November 1st, 1910; died December 14th, 1913.

Johannes Cornelis Vliegenthart, of Delft, Holland, elected Associate Member, June 5th, 1907; died in October, 1913.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

December 31st, 1913.—Vice-President Churchill in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Bates, Bush, Cain, Clarke, Edwards, Hodge, Leonard, Ridgway, and Snow.

Details of the Report of the Board of Direction to the Society were discussed.

A Report was received from the Committee appointed to Recommend the Award of Medals and Prizes, Messrs. Charles M. Spofford, Daniel W. Mead, and Walter Loring Webb, and the Board awarded the Prizes as follows:

The Norman Medal to Paper No. 1235, entitled "Air Resistances to Trains in Tube Tunnels," by J. V. Davies, M. Am. Soc. C. E.

The Thomas Fitch Rowland Prize to Paper No. 1231, entitled "The Laramie-Poudre Tunnel," by Burgis G. Coy, Assoc. M. Am. Soc. C. E.

The J. James R. Croes Medal to Paper No. 1222, entitled "The Problem of the Lower West Side Manhattan Water-Front of the Port of New York," by B. F. Cresson, Jr., M. Am. Soc. C. E.

The James Laurie Prize to Paper No. 1218, entitled "Construction of the Morena Rock Fill Dam, San Diego County, California," by M. M. O'Shaughnessy, M. Am. Soc. C. E.

Messrs. George W. Tillson, Nelson P. Lewis, and Charles J. Tilden, were appointed as additional members of the Special Committee on Bituminous Materials for Road Construction.

Action was taken in regard to members in arrears for dues.

The Constitution adopted by the Texas Association of Members of the Society was approved by the Board.

The resignations of 7 Members, 7 Associate Members, 3 Associates, and 8 Juniors, were accepted.

Ballots for membership were canvassed, resulting in the election of 13 Members, 25 Associate Members, and 11 Juniors, and the transfer of 10 Juniors to the grade of Associate Member.

Seven Associate Members were transferred to the grade of Member, and one Associate was transferred to the grade of Associate Member.

Applications were considered, and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

February 4th, 1914.—8.30 P. M.—A regular business meeting will be held. A paper by Edward Flad, M. Am. Soc. C. E., entitled "Reinforced Concrete Reservoir and Coagulation Plant at St. Louis, Mo.," will be presented for discussion, and George W. Tillson, M. Am. Soc. C. E., will address the meeting on "Some Observations on Pavements in Foreign Cities."

Mr. Flad's paper was printed in *Proceedings* for December, 1913.

February 18th, 1914.—8.30 P. M.—Two papers will be presented for discussion at this meeting, as follows: "Grouted Cut-Off for the Estacada Dam," by Harold A. Rands, Assoc. M. Am. Soc. C. E.; and "Diversion of Irrigating Water from Arizona Streams," by A. L. Harris, Assoc. M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

SPECIAL COMMITTEE TO REPORT ON STRESSES IN RAILROAD TRACK

At the Society meeting of June 4th, 1913, the following motion was offered:

"That it is the sense of this meeting that the American Society of Civil Engineers should appoint a Special Committee to act jointly with the American Railway Engineering Association in studying and experimenting on the stresses in railroad rails, ties, etc."

This motion was referred to the Board of Direction.

The Board has appointed the following Committee:

A. N. Talbot, *Chairman,*

A. S. Baldwin,	Emil Gerber,
J. B. Berry,	Robert W. Hunt,
G. H. Bremner,	George W. Kittredge,
John Brunner,	William McNab,
W. J. Burton,	G. J. Ray,
Charles S. Churchill,	F. E. Turneaure,
W. C. Cushing,	J. E. Willoughby.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

**Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Visiting members are urged to attend the meetings.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and B. M. Hall, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

Philadelphia Association

On December 22d, 1913, the Philadelphia Association of Members of the American Society of Civil Engineers was organized, with the following officers: George S. Webster, President; Richard L. Humphrey and F. Herbert Snow, Vice-Presidents; J. W. Ledoux, Edgar Marburg, and H. S. Smith, Directors; S. M. Swaab, Treasurer; and W. L. Stevenson, Secretary. The meetings of the Association will be held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

Seattle Association

On June 30th, 1913, the Seattle Association of Members of the American Society of Civil Engineers was organized with the following officers: Samuel H. Hedges, President; Ernest B. Hussey, Vice-President; and Joseph Jacobs, Secretary-Treasurer.

Southern California Association

At its meeting of December 3d, 1913, the Board of Direction considered and approved the proposed Constitution of the Southern California Association of Members of the American Society of Civil Engineers.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, Room 6, City Bank and Trust Company Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31ST, 1913.

In compliance with the Constitution, the Board of Direction presents its report for the year ending December 31st, 1913.

MEMBERSHIP

The changes in membership are shown in the following table:

	Jan. 1st, 1913.			Jan. 1st, 1914.			Losses.				Additions.		Total.		
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.	Increase.
Honorary Members.	.....	7	7	.....	5	5	....	..	..	3	1	.....	3	1	‖2
Corresponding " .	.....	2	2	.....	1	1	....	..	..	1	.....	.....	1	....	‖1
Members.	633	2 432	3 065	648	2 627	3 275	1	13	9	39	*125	§147	62	272	210
Associate Members.	513	2 178	2 691	547	2 437	2 984	120	15	11	13	†114	††338	159	452	293
Associates	76	103	179	77	97	174	7	4	1	3	‡3	7	15	10	‖5
Juniors	156	658	814	156	660	816	115	13	19	2	.....	151	149	151	2
Fellows	7	12	19	5	12	17	....	..	..	2	.....	.....	2	....	‖2
Totals	1 385	5 392	6 777	1 433	5 839	7 272	243	45	40	63	243	643	391	886	495

* 120 Associate Members and 5 Associates.
§ 2 Reinstatements.
† 2 Associates and 112 Juniors.
†† Includes 9 Juniors dropped on account age limit in 1912 and 2 Reinstatements.
‡ 3 Juniors.
‖ Decrease.

It will be noted that the net increase in membership for the year is 495.

The number of applications received during 1913 was 1 096: 803 for admission, and 293 for transfer.

The losses by death reported during the year number 63, and are as follows:

Honorary Members (3): John Fritz, George Edward Gray, *Sir* William Henry White.

Corresponding Member (1): Ernest Pontzen.

Members (39): Arthur Lincoln Adams, Peter Suther Archibald, James Richard Bell, Adolphus Bonzano, Albert Safford Cheever, Philip Henry Coombs, Samuel Lispenard Cooper, Frederic Danforth, John Butler Duncklee, Griffith Morgan Eldridge, Rudolph Fink, John Douglas Fouquet, George Blinn Francis, William Gaston Hamilton, James Charles Haugh, Horace Theophilus Herrick, Franklin Allen Hinds, Ned Herbert Janvrin, Charles Kellogg, Emil Edward Kuersteiner, Henry Francis Labelle, Francis Valentine Toldervy Lee,

Ulysses Stanislaus Lutz, David Ernest Melliss, John Wiley Miles, Alonzo Tyler Mosman, George Browne Post, William Napier Radenhurst, Charles Walker Raymond, James Ross, Robert Wilson Sayles, Baird Snyder, Jr., Frank Soulé, David McNeely Stauffer, Henry William Vehrenkamp, Ebenezer Smith Wheeler, Henry Shotwell Wood, Emanuel Alois Ziffer, Luther Reese Zollinger.

Associate Members (13): Charles De La Plane Atterbury, James Marcus Bandy, Arthur Garfield Crysler, John Stuart Elliott, Harry Woy Gray, Henry Alexander Harris, George Gere MacCracken, Horace Guy Merrick, William Belden Reed, Jr., Charles Spearman, Charles Harry Tisdale, Alberto de la Torre, Johannes Cornelis Vliegenthart.

Associates (3): Samuel Stockton Bogart, Alfred G. Compton, John MacGregor.

Juniors (2): Henry Helm Clayton, Orloff Lake.

Fellows (2): Henry A. Richmond, James Watson.

LIBRARY

The total contents of the Library and the increase during the year, are shown in the following statement:

	Total Contents.	Increase during 1913.
Bound volumes	23 357	1 450
Unbound volumes	44 405	2 273
Specifications	7 420	223
Maps, photographs and drawings	4 674	76
Total	79 856	4 022

Of these, 1 732 were donations received in answer to special requests; 91 were donations from publishers; 1 867 were donations received in regular course, and 332 were purchased.

The value of accessions to the Library during the year is as follows, each accession having been valued separately as received:

Donations and exchanges (estimated value)	$2 625.70
332 volumes purchased (cost)	1 004.86
Binding 428 volumes	496.63
Total	$4 127.19

The following amounts have been expended upon the Library during the year:

Purchases, subscriptions, and binding	$1 501.49
Fixtures, supplies, express charges, etc.	186.76
Total	$1 688.25

The card index now contains about 87 000 cards.

During the year 79 new bibliographies (containing 3 258 separate references) have been made, copies of 35 searches made in previous years have been furnished, 4 of these having been brought up to date. The total cost of this work, $1 025.83, has been charged to those for whom it was undertaken.

The total attendance in the Reading Room and Library during the year, which shows a marked increase over that of previous years, was 4 887. This does not include those who use the Library during the semi-monthly meetings.

SPECIAL COMMITTEES

Much thought has been given during the year to the work of Special Committees appointed to report on Engineering Subjects. Some time ago the Constitution was amended, in order that, when expedient, Special Committees might be appointed promptly. Your Board, believing that through the work of such Committees the Society can enlarge its usefulness to the Profession, has appointed four new Special Committees during the year, and has indicated the willingness and ability of the Society to furnish greater financial aid to this work than has heretofore been possible.

The new Committees appointed were the following:

To Codify Present Practice on the Bearing Value of Soils for Foundations.

On A National Water Law.

On Floods, Flood Prevention, and other Allied Subjects.

On Stresses in Railroad Track.

In addition to these four Special Committees, there are six others previously appointed, from all of which Progress Reports have been received and issued to members during the year. These Committees are on the following subjects:

On Concrete and Reinforced Concrete.

On Engineering Education.

On Steel Columns and Struts.

On Bituminous Materials for Road Construction.

On Valuation of Railroad Property and other Public Utilities.

To Investigate Conditions of Employment of, and Compensation of, Civil Engineers.

PUBLICATIONS

During the year, ten numbers of *Proceedings,* one Volume of *Transactions,* and one List of Members, have been issued.

In *Proceedings* the list of references to current engineering literature has been continued, and has covered 160 pages and contained 7 206 classified references to 107 periodicals.

The stock of the various publications of the Society, kept on hand for the convenience of members and others, now amounts to 167 338 copies, the cost of which to the Society, for paper and press work only, has been $25 612.41.

During the year, 4 595 volumes of *Transactions* have been bound for members and others in standard half-morocco and cloth bindings.

SUMMARY OF PUBLICATIONS FOR 1913.

	Issues.	Average Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volume LXXVI)..	1	7 700	2 302	76	655
Proceedings (Monthly numbers)..	10	7 625	3 184	104	497
Constitution and List of Members.	1	7 850	296	....	1
Total	12	....	5 782	180	1 153

The cost of publications has been:

For Paper, Printing, etc., *Transactions* and *Proceedings*....	$38 698.05
For Plates and Cuts..................................	3 964.16
For Boxes, Mailing Lists, Copyright, and Sundry Expenses..	667.56
For 41 150 Extra Copies of Papers and Memoirs..........	2 106.31
For List of Members..................................	1 964.84
Total	$47 400.92
Deduct amount received from sale of publications........	4 908.46
Net expenditure for publications for 1913..........	$42 492.46

Volume LXXVI of *Transactions* issued in December, 1913, has been received by the Membership with much favorable comment. It is the second of the yearly volumes printed on Bible paper, and contains 2 300 pages. The advantages of this form of publication over that which necessitated the issue of four volumes for the same amount of material—viz.: the saving of shelf room, one index instead of four, greater length of time which can be allowed for closing of discussions, economy to members in binding and to the Society in postage, have been indicated in previous reports.

Beginning with the April Number, the *Proceedings* have also been printed on a thin paper, and the advantages of this change have also been evident. It will be noted that the average number of pages in each number is 318 and that some issues are much larger even than this. When printed on the heavier paper a pamphlet of from 350 to 400 pages becomes a volume which is too bulky and heavy for convenient handling.

MEETINGS

Twenty-six meetings were held during the year, as follows: At the Annual Meeting, 1; at the Annual Convention, 3; and 22 other meetings, 21 of which were held at the Society House.

The experiment of holding one of the regular meetings at a point outside of the Resident District was tried, and the meeting scheduled for October 15th, held in New Orleans, La., was a success in every way.

At these meetings there were presented 29 formal papers, 6 of which were illustrated with lantern slides. There were also 12 papers published which were not presented for discussion at any meeting of the Society. The number of members and others who took part in the preparation or discussion of these papers was 275.

The Forty-fifth Annual Convention was held at Ottawa, Ont., Canada.

The total attendance at the 26 meetings held was about 5 220. The registered attendance at the Annual Meeting was more than 1 200, at the Annual Convention 138, at the New Orleans Meeting 121 (includes members only), but there were many guests present at all these meetings, and also Members who failed to register.

At each of the ordinary semi-monthly meetings held during the year Collations have been served, and these have been paid for out of the Society Funds, in accordance with the action of the Annual Meeting of 1912.

MEDALS AND PRIZES

For the year ending with the month of July, 1912, prizes were awarded as follows:

The Norman Medal to Wilson Sherman Kinnear, M. Am. Soc. C. E., for his paper entitled "The Detroit River Tunnel."

The Thomas Fitch Rowland Prize to Eugene Klapp and W. J. Douglas, Members, Am. Soc. C. E., for their paper entitled "Reinforced Concrete Bridge Across the Almendares River, Havana, Cuba."

The Collingwood Prize for Juniors to W. W. Clifford, Jun. Am. Soc. C. E., for his paper entitled "A Reinforced Concrete Stand-Pipe."

FINANCES

The mortgage debt of the Society has been reduced during the year $35 000, from $115 000 to $80 000, and the Reserve Fund now consists of $80 000 (par value) New York City Bonds, on which the interest received is a little more than that paid on the mortgage, therefore the Society may be said to be for the first time out of debt.

Appended to this Report are diagrams showing for the last 32 years the following: Total Membership; Total Number of Pages Published; Total Salaries; Cost of Publications; Receipts for Publica-

tions; Expenditures Charged to Library; Receipts per Member; Expenditures per Member; Total Salaries per Member; Net Cost of Publications per Page per Member; and a general curve showing the Ratio of Expenditures to Receipts.

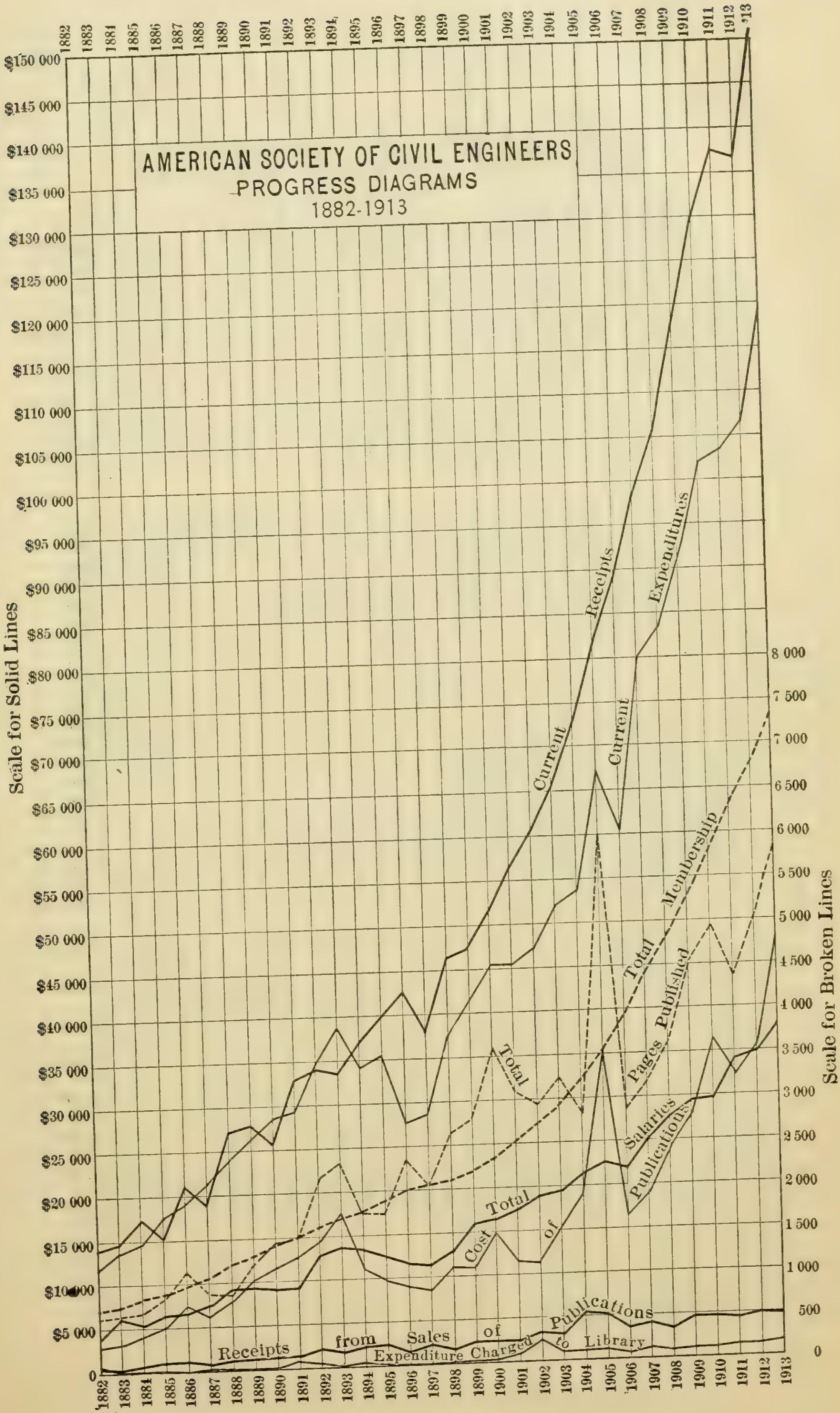
The attention of members is invited to the Secretary's statement of receipts and disbursements, and to the general balance sheet which accompanies it, in which the financial condition of the Society is shown.

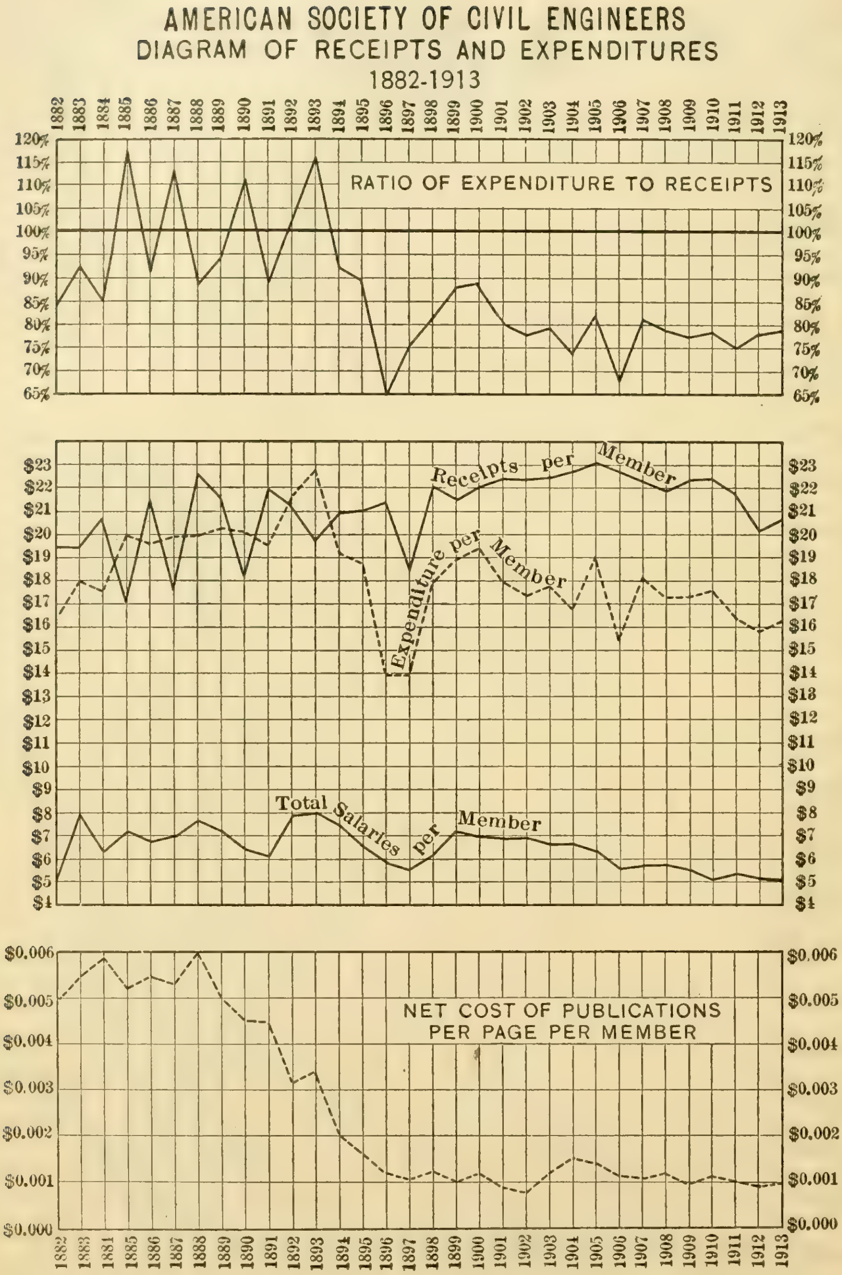
The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction.

CHAS. WARREN HUNT,
Secretary.

JANUARY 7TH, 1914.


AMERICAN SOCIETY OF CIVIL ENGINEERS
PROGRESS DIAGRAMS
1882-1913
Scale for Solid Lines
Scale for Broken Lines
Current Receipts
Current Expenditures
Total Membership
Total Pages Published
Salaries
Publications
Total Cost of Publications
Receipts from Sales of Publications
Expenditure Charged to Library


AMERICAN SOCIETY OF CIVIL ENGINEERS
DIAGRAM OF RECEIPTS AND EXPENDITURES
1882-1913
RATIO OF EXPENDITURE TO RECEIPTS
Receipts per Member
Expenditure per Member
Total Salaries per Member
NET COST OF PUBLICATIONS
PER PAGE PER MEMBER

GENERAL BALANCE SHEET, DECEMBER 31st, 1913.

Accompanying the Report of the Secretary.

Assets.			Liabilities.	
Three Lots (Actual cost, $189 632.11) (Estimated Value)		$375 000.00	Dues for 1914 paid in advance	$30 326.25
Society Building (cost)		170 955.59	Mortgage Debt and Loan	80 000.00
Furniture (cost)		20 663.60	Funds invested in Society House, Lots and Library*	27 190.78
Publications on hand (inventoried cost)		25 612.41	Herbert Stewart Library Fund	1 997.50
New York City Non-Taxable Bonds (cost)		80 425.00	Gen. Joseph G. Swift Library Fund	998.75
Library: Cash expended for books, etc.	$18 626.84		Surplus (including Reserve Fund of $77 428.75)	644 829.57
Donations (estimated)	65 433.69	84 060.53		
Due from Members		9 475.94		
Due from Non-Members		716.35		
Cash		18 433.43		
		$785 342.85		$785 342.85

We have examined the books and accounts of the American Society of Civil Engineers, for the year ended December 31, 1913, and certify that the foregoing Balance Sheet is in accordance therewith, and, in our opinion, correctly states the condition of the Society's affairs, as shown by the books.

79 Wall Street, New York.
January 7th, 1914.

Marwick, Mitchell, Peat & Co.,
Chartered Accountants.

*Compounding Dues Fund, $10 930.00; Norman Medal Fund, $1 000.00; Rowland Prize Fund, $1 222.50; Collingwood Prize Fund, $1 000.00; Fellowship Fund, $13 038.28.

REPORT OF THE SECRETARY FOR THE

To the Board of Direction of the

Gentlemen:—I have the honor to present a statement of Receipts 31st, 1913. I also append a general balance sheet showing the condition

Receipts.

Balance on hand December 31st, 1912, in Bank, Trust Company, and in hands of Treasurer		$44 074.04
Entrance Fees	$16 680.00	
Current Dues	78 971.31	
Past Dues	3 277.18	
Advance Dues	30 326.25	
Certificates of Membership	754.30	
Badges	3 385.50	
Sales of Publications	4 908.46	
Library	672.13	
Annual Meeting	798.65	
Binding	6 489.36	
Interest	4 509.03	
Miscellaneous	311.33	
		151 083.50
		$195 157.54

YEAR ENDING DECEMBER 31st, 1913.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

and Disbursements for the fiscal year of the Society, ending December of the affairs of the Society.

Respectfully submitted,
CHAS. WARREN HUNT.
Secretary.

DISBURSEMENTS.

Salaries of Officers	$14 550.00	
Mileage of Directors	948.96	
Mileage of Nominating Committee	618.90	
Work of Committees	793.67	
Clerical Help	21 192.82	
Caretaking	1 905.24	
Publications	47 400.92	
Postage	7 376.40	
General Printing and Stationery	4 119.07	
Library	1 501.49	
Library Maintenance	186.76	
Badges	2 330.25	
Certificates of Membership	569.60	
Binding	3 210.46	
Prizes	443.05	
Annual Convention	391.12	
Annual Meeting	1 818.00	
Maintenance of House	700.49	
Heat, Light, and Water	867.53	
Betterments	118.30	
Furniture	552.95	
Interest	4 321.80	
Insurance	290.00	
Current Business	2 562.39	
Petty Expenses	215.71	
International Engineering Congress, 1915	2 730.06	
Advanced on Members' Accounts	20.67	
Bond and Mortgage (Payment on Principal)	35 000.00	
Reserve Fund (Purchase of Bonds)	19 987.50	
		$176 724.11
Balance on hand December 31st, 1913:		
In Union Trust Company	$10 158.57	
In Garfield National Bank	6 774.86	
In hands of Secretary	1 500.00	
		18 433.43
		$195 157.54

REPORT OF THE TREASURER.

In compliance with the provisions of the Constitution, I have the honor to present the following report for the year ending December 31st, 1913:

Balance on hand December 31st, 1912			$44 074.04
Receipts from current sources, January 1st to December 31st, 1913			151 083.50
Payment of Audited Vouchers for Current Business, January 1st to December 31st, 1913		$121 736.61	
Payment on principal of bond and mortgage		35 000.00	
Purchase of bonds, Reserve Fund		19 987.50	
Balance on hand December 31st, 1913:			
In Union Trust Company	$10 158.57		
In Garfield National Bank	6 774.86		
In hands of the Secretary	1 500.00	18 433.43	
		$195 157.54	$195 157.54

Respectfully submitted,

JOHN F. WALLACE,
Treasurer.

NEW YORK, JANUARY 7TH, 1914.

ACCESSIONS TO THE LIBRARY

(From December 2d, 1913, to January 5th, 1914)

DONATIONS*

SEWER DESIGN.

By H. N. Ogden, M. Am. Soc. C. E. Second Edition. Cloth, $9\frac{1}{4}$ x 6 in., illus., 13 + 248 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1913. $2.00.

The first edition of this work was issued fifteen years ago, and represented a course of lectures given by the author in the College of Civil Engineering at Cornell University. The book has been used since that time by the author in his classes, and although, as a whole, the subject-matter has not been materially changed in this, the second edition, certain errors, it is said, have been corrected and additions have been made in order to bring it up to present practice. It contains, it is stated, all the essentials of sewer design for a general course in Civil Engineering arranged so as to be readily intelligible to the average student. At the end of each chapter problems relating to the subject discussed in that chapter are given, in order that the student may apply his theory to concrete examples. Numerous references are also given to original papers in connection with the several subjects. The Chapter headings are: General Considerations; Preparatory Maps and Data; Excessive Rains; Proportion Reaching the Sewers; Relation of Density to Percentage; Mathematical Formulas; Estimating Future Population; Amount of Sewage per Capita; Ground-Water Reaching Sewers; Grades and Self-Cleansing Velocities; Development of Formulas for Flow; Kutter's Formula; Sewer Diagrams; Use of Diagrams; Sewer Plans; Sewer Cross-Sections; Flushing; Use of Tanks; Index.

THE GAS-ENGINE HANDBOOK:

A Manual of Useful Information for the Designer and the Engineer. By E. W. Roberts. Seventh Edition, Entirely Rewritten and Enlarged. Morocco, 7 x $4\frac{1}{2}$ in., illus., 315 pp. Cincinnati, Ohio, The Gas Engine Publishing Co., 1913. $2.00.

The preface states that this Handbook has been written as an epitome of gas-engine practice, and is intended for use as a handy book of reference. As it is not an extended treatise on the subject, the author, it is said, has devoted the subject-matter to that which he has judged to be most useful and which he has had frequent occasion to look up himself. He has divided the book into three parts, in the first of which he describes methods and devices used in gas-engine practice, as well as motors for automobiles, boats, and aeroplanes. The second part is devoted to the design of gas engines. In this part the subject-matter has been brought up to date, and new formulas of design and new tables have been added. In the third part, the subjects of operation, testing, and selection are discussed, in which apparatus and methods are described and suggestions for the selection of engines for various requirements given. The Contents are: Part I, Descriptive: Introductory; Comparison of the Cycles; Gas-Engine Fuels; Mechanism of the Gas Engine; Valves; Gas Valves and Mixers; Carbureters and Gasoline Mixers; Oil Engine Vaporizers and Mixers; Make-and-Break Igniters; Jump-Spark Ignition; Miscellaneous Ignition Systems; Gas-Engine Governors; Starting Large Engines; Engines for Automobiles; Marine Motors; Aeronautical Motors; Lubrication. Part II, Design: The Indicator Diagram; General Dimensions; The Cylinder; Pistons, Rods and Shafts; Valves; Igniters; Frames and Bases; Flywheels; Governor Design; Balancing; Foundations; Mufflers; Two-Cycle Engines; Aeroplane Engines. Part III, Operation, Testing, Selection: Installation; Starting and Stopping; Care of Gas Engines; Troubles; Testing; Selection; Tables of Dimensions of Gas and Water Pipe; Areas and Circumferences of Circles; Strength of Materials; Specific Gravity of Metals; Index.

DESCRIPTIONS OF LAND:

A Text-Book for Survey Students. By R. W. Cautley. Cloth, $7\frac{3}{4}$ x $5\frac{1}{4}$ in., illus., 9 + 89 pp. New York, The Macmillan Company, 1913. $1.00.

In drafting a description of lands a surveyor must have, the Introduction states, a clear conception of what he has to describe, of the surveying operations involved,

*Unless otherwise specified, books in this list have been donated by the publishers.

and of the exact meaning of the words he is using, as interpreted by judicial decisions. He must also be able to express what he wishes to state clearly and accurately. As all students of surveying in Canada are required to pass an examination in "descriptions of land," and as all titles to land are based on the description contained in the deed, the author, it is said, has written this textbook with the hope that it will prove of use to such students in their examinations and aid them in formulating rules for writing the descriptions required. The Contents are: Descriptions Based on Actual Survey; Descriptions Must Refer to Plans of Record; Plans and How to Refer to Them; The Use of Natural Boundaries in Descriptions; The Use of the Words "More or Less" in Descriptions; The Use of Astronomical Bearings in Descriptions; Description of Remainders; Description by Exception; Description of Railway Right of Way; Exception of Minerals in Descriptions; Interpretation of Faulty Descriptions; Forms of Preamble; Description of Examples.

CITY-PLANNING:

A Comprehensive Analysis of the Subject Arranged for the Classification of Books, Plans, Photographs, Notes and Other Collected Material, with Alphabetic Subject Index. By James Sturgis Pray and Theodora Kimball. Paper, 10 x 7¼ in., 103 pp. Cambridge, Mass., Harvard University Press, 1913. 50 cents. (Donated by the Harvard University School of Landscape Architecture.)

The Classification contained in this work is stated to be the first comprehensive, systematic setting forth of the scope and varied character of this field. The newness of city planning (which includes the planning and organization of towns, suburbs, villages, and related rural districts) as an art, science, and profession, makes, it is said, an even development of the classification scheme impossible, but it is hoped that the systematic handling of the subject, as developed by the authors, may prove useful either for the classification of collected material, for the indication of relations between special parts of the whole field, or for a systematic analysis of the subject. It is intended for municipal officials, engineers, architects, students of city planning, librarians, etc., and should prove useful, it is stated, for classifying collections great or small, general or special. The Classification was originally developed for the material on city planning contained in the Library of the School of Landscape Architecture, at Harvard University, and is easily adaptable to other systems of classification. Its use is facilitated by an alphabetical subject index, and the authors, it is stated, have taken every means to make this index of the greatest possible service to all users of the Classification.

HANDBOOK ON SANITATION:

A Manual of Theoretical and Practical Sanitation. By George M. Price. Third Edition, Rewritten and Reset. Cloth, 7¾ x 5¼ in., illus., 11 + 353 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1913. $1.50.

In a secondary title, it is stated that this handbook is intended for students and physicians, for health, sanitary, tenement-house, plumbing, factory, food, and other inspectors, as well as for candidates for all State and municipal sanitary positions, and the author hopes that the subject-matter may be of use to all who are interested in municipal sanitation and sanitary science. The first edition of this work, which is said to have been practically the first of its kind in the United States, was published in 1901, and this, the third, edition has been considerably revised and brought up to date, the chapter on "Ventilation" and the section on "Foods" having been entirely rewritten and considerably enlarged. Under the chapter on "Civil Service Examinations" are the questions given to candidates for various sanitary positions, by the New York State and Municipal Civil Service Commissions, a study of which it is hoped may prove beneficial to candidates and students in general. The Contents are: Part I, Sanitary Science: Soils and Sites; Air; Ventilation; Warming; Water; Water-Supply; Disposal of Sewage; Sewers; Plumbing: General Principles; Plumbing Pipes; Plumbing Fixtures; Defects in Plumbing Examination and Tests. Part II, Sanitary Practice: Section 1, Habitation; Section 2, Occupations and Trades; Section 3, Foods; Section 4, Disinfection and Disinfectants. Part III, Sanitary Inspection: Sanitation as a Profession; Qualifications For and Art of Inspection; Tenement-House Inspection; Civil-Service Examinations; Calculation of Areas and Cubic Space; Useful Memoranda and Tables; Index.

A READER OF SCIENTIFIC AND TECHNICAL SPANISH

For Colleges and Technological Schools, with Vocabulary and Notes. By Cornélis DeWitt Willcox. Cloth, $7\frac{3}{4}$ x $5\frac{1}{4}$ in., illus., 588 pp. New York, Sturgis & Walton Company, 1913. $1.75.

The author states that this work has been prepared in the belief that it may be of use to students in colleges and technical schools who mean to practice engineering in Spanish-speaking Americas. The subject-matter consists of definitions and extracts from Spanish works relating to many branches of practical science, together with illustrations and a few notes, and, as presented, it is thought that it will give the reader not only a fair idea of the Spanish technical vocabulary, but, within the limits of the text, even a working knowledge of it. It is assumed, it is stated, that the student is already acquainted with ordinary Spanish, and the vocabulary, therefore, is limited to the scientific and technical words occurring in the text selected. The Contents are: Física; Química; La Conservación de las Pólvoras Modernas; Electricidad; Utilización del Agua Como Fuerza Motriz; Vapor; Aplicaciones del Aire Comprimido; Turbina; Transporte de la Fuerza á Distancia; Minería; Puentes; Ferrocarriles; Agrimensura; Topografía; Geografía; El Automóvil; Aeronáuticas; Submarinos; La Campaña de Santiago de Cuba; Vocabulario.

Gifts have also been received from the following:

Aberdeen, Scotland-Water Engr. 1 pam.
Abrams, D. A. 1 pam.
Allison, James F. 1 vol., 4 pam.
Alvord & Burdick. 1 vol.
Am. Ceramic Soc. 1 vol.
Am. Inst. of Min. Engrs. 1 vol.
Am. Water Works Assoc. 8 pam.
Arnold, Bion J. 1 bound vol.
Ayr, Scotland-Town Clerk. 1 pam.
Baltimore, Md.-Harbor Board. 1 pam.
Baltimore & Ohio R. R. Co. 1 pam.
Barrow-in-Furness, England-Town Clerk. 1 bound vol.
Bethnal Green (London), England-Town Clerk. 1 bound vol.
Birmingham, England-Water Dept. 2 pam.
Bolton, England-Borough Engr. and Surv. 8 bound vol.
Bournemouth, England-Town Clerk. 2 pam.
Bradford, England-Town Clerk. 1 vol.
Brooklyn, N. Y.-Supt. of Bldgs. 4 pam.
Cedar Rapids, Iowa-City Clerk. 2 pam.
Chicago, Ill.-Bureau of Public Efficiency. 1 pam.
Chicago, Ill.-Dept. of Public Works. 1 pam.
Coatbridge, Scotland-Town Clerk. 1 pam.
Colorado Springs, Colo.-Water Dept. 1 pam.
Columbus, Ohio-Chf. Engr. of Dept. of Public Impvts. 1 vol.
Council Bluffs, Iowa-City Auditor. 2 pam.
Darlington, England-Town Clerk. 2 pam.
Datesman, George E. 1 vol.
Dooling, Peter J. 1 pam.
Eastbourne, England-Town Clerk. 1 pam.
Ferrocarriles Nacionales de Mexico. 1 pam.
Finsbury (London), England-Town Clerk. 1 vol.
Fitchburg, Mass.-Board of Water Commrs. 1 vol., 1 pam.
Ford, Bacon & Davis. 2 bound vol.
Fulham, England-Town Clerk. 1 bound vol.
Germany-Kaiserliche Generaldirektion der Eisenbahnen in Elsass-Lothringen. 1 vol.
Glasgow, Scotland-Town Clerk. 3 pam.
Hackney (London), England-Town Clerk. 1 bound vol., 2 pam.
Halifax, England-Town Clerk. 1 pam.
Hammersmith (London), England-Town Clerk. 1 bound vol.
Harrogate, England-Town Clerk. 1 pam.
Houston, Tex.-Water Commr. 2 pam.
Illinois-State Geol. Survey. 1 vol.
India-Geol. Survey. 1 vol.
Institution of Gas Engrs. 1 bound vol.
Inst. of Min. Engrs. 1 pam.
Iowa-Auditor of State. 1 bound vol.
Iowa-Geol. Survey. 1 bound vol.
Iowa State Coll. of Agri. and Mechanic Arts. 1 pam.
Ipswich, England-Water Engr. 3 pam.
Kalamazoo, Mich.-City Clerk. 1 bound vol.
Kansas City, Kans.-Finance Commr. 1 pam.
Kingston-upon-Hull, England-Medical Officer of Health. 1 bound vol., 1 pam.
Leeds, England-Water-Works Engr. 3 pam.
Luton, England-Town Clerk. 2 pam.
Madison, Wis.-City Clerk. 20 pam.
Madras, India-Public Works Dept. 1 bound vol.
Maryland-State Board of Health. 1 vol.
Mass. Inst. of Tech. 1 vol.
Matschoss, Conrad. 1 pam.
National Board of Fire Underwriters. 13 pam.
National Fire Protection Assoc. 3 pam.
New Bedford, Mass.-Clerk of Common Council. 1 bound vol.
New Jersey-Geol. Survey. 2 pam.
New London, Conn.-Water and Sewer Commrs. 1 pam.
New York City-Bureau of Franchises. 1 pam.
New York City-Chf. Engr. of Board of Estimate and Apportionment. 1 pam.
New York State-Comm. to Investigate Port Conditions and Pier Extensions in New York Harbor. 1 pam.
New York-State Conservation Comm. 1 bound vol.
New York City Record. 1 bound vol.
Newcastle & Gateshead (England) Water Co. 1 pam.
Newport, Ky.-Water-Works Dept. 1 pam.

North, Arthur T. 1 pam.
Nottingham, England-Town Clerk. 2 bound vol.
Nottingham, England-Water Dept. 3 pam.
Ogden, Utah-City Auditor. 1 pam.
Ontario Canada-Provincial Board of Health. 1 pam.
Ontario, Canada-Registrar-Gen. 1 vol.
Oregon, Univ. of. 1 vol.
Paddington, England-Town Clerk. 1 bound vol.
Pennsylvania-Topographic and Geol. Survey. 2 bound vol.
Pennsylvania Lines West of Pittsburgh. 1 bound vol.
Pere Marquette R. R. Co. 2 pam.
Philippine Islands-Bureau of Customs. 1 pam.
Philippine Islands-Bureau of Forestry. 1 pam.
Philosophical Soc., Univ. of Virginia. 1 pam.
Plymouth, England-Town Clerk. 3 pam.
Plymouth, Mass.-Water Commrs. 5 pam.
Poplar, England-Town Clerk. 1 pam.
Porto Rico-Dept. of Sanitation. 1 vol.
Portsmouth, England-Town Clerk. 1 bound vol.
Puff, Charles F., Jr. 1 pam.
Ramsgate, England-Town Clerk. 4 pam.
Reading, England-Town Clerk. 1 pam.
Rochdale, England-Town Clerk. 1 bound vol.
St. Pancras (London), England-Town Clerk. 1 bound vol.
Saskatchewan, Canada-Board of Highway Commrs. 1 pam.
Saskatchewan, Canada-Dept. of Public Works. 1 pam.
Shoreditch (London), England-Town Clerk. 1 bound vol.
Southend-on-Sea, England-Borough Engr. 1 pam., 1 map.
Spring Valley Water Co. 2 pam.
Staniford, Charles W. 1 pam.
Texas & Pacific Ry. Co. 1 pam.
Todmorden, England-Town Clerk. 1 pam.
Toronto, Ont.-Mayor. 2 pam.
Tynemouth, England-Town Clerk. 1 bound vol.
Tyrrell, Henry Grattan. 2 pam.
Ulster & Delaware R. R. Co. 1 pam.
Union Univ. 1 vol.
U. S.-Bureau of Navigation. 2 pam.
U. S.-Census Bureau. 1 bound vol., 1 pam.
U. S.-Chf. of Engrs. 13 specif.
U. S.-Chf. Signal Officer. 2 pam.
U. S.-Coast and Geodetic Survey. 1 pam.
U. S.-Dept. of Commerce. 1 vol.
U. S.-Interstate Commerce Comm. 1 bound vol.
U. S.-Isthmian Canal Comm. 1 bound vol., 1 pam., 62 maps.
U. S.-Lake Survey Office. 1 map.
U. S.-Library of Congress. 1 bound vol., 1 pam.
U. S.-National Museum. 1 vol.
U. S.-Navy Dept. 1 pam.
U. S.-Office of Naval Intelligence. 2 pam.
U. S.-Office of Public Roads. 1 pam.
U. S.-Public Health Service. 1 bound vol.
Victoria, Australia-Dept. of Mines. 1 vol.
Wakefield, England-Water-Works Engr. 3 pam.
Wallasey, England-Town Clerk. 1 pam.
Wallsend, England-Town Clerk. 1 bound vol.
West Virginia, Univ. of. 1 bound vol., 2 vol.
Wimbledon, England-Town Clerk. 5 pam.
Wisconsin-State Board of Health. 1 vol.
Work, John D. 2 pam.

BY PURCHASE

Seehafenbau. Von F. W. Otto Schulze. Band 2, Ausbau der Seehäfen. Wilhelm Ernst & Sohn, Berlin, 1913.

Handbuch der Ingenieurwissenschaften: Dritter Teil, Der Wasserbau: Zweiter Band, Zweite Abteilung: Die Talsperren. Von E. Mattern, herausgegeben von Th. Rehbock. Vierte, vermehrte Auflage. Wilhelm Engelmann, Leipzig und Berlin, 1913.

Hydraulique. Par A. Flamant. Troisieme Edition, Revue et Augmentée. Librairie Polytechnique, Ch. Beranger, Editeur, Paris, 1909.

The Surveyor and Municipal and County Engineer, Vol. 17, 18, 20, 23, 24, and 30, for the Years 1900, 1901, 1903, and 1906. The St. Bride's Press, Ltd., London.

Logging: The Principles and General Methods of Operation in the United States. By Ralph Clement Bryant. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1913.

Handbuch für Eisenbetonbau: Erster Band, Entwicklungsgeschichte und Theorie des Eisenbetons: Zweite Auflage, Die Grundzüge

der Geschichtlichen Entwicklung des Eisenbetons: Theorie und Versuche. Von M. Foerster and others. Wilhelm Ernst & Sohn, Berlin, 1912.

Quantitative Analysis by Electrolysis. By Alexander Classen, with the Coöperation of H. Cloeren. Translated from the Thoroughly Revised Fifth German Edition by William T. Hall. John Wiley & Sons, New York; Chapman & Hall, Ltd., London, 1913.

Beiträge zur Geschichte der Technik und Industrie: Jahrbuch des Vereines Deutscher Ingenieure, Von Conrad Matschoss. Vol. 1–3. Julius Springer, Berlin, 1909-11.

Metallurgical Analysis. By Nathaniel Wright Lord and Dana J. Demorest. Third Edition. McGraw-Hill Book Co., New York and London, 1913.

Mineral Deposits. By Waldemar Lindgren. McGraw-Hill Book Co., New York and London, 1913.

Refractories and Furnaces: Properties, Preparation, and Application of Materials Used in the Construction and Operation of Furnaces. By F. T. Havard. McGraw-Hill Book Co., New York and London, 1912.

Introduction to the Study of Igneous Rocks. By George Irving Finlay. McGraw-Hill Book Co., New York and London, 1913.

SUMMARY OF ACCESSIONS

(From December 2d, 1913, to January 5th, 1914)

Donations (including 13 duplicates)	255
By purchase	19
Total	274

MEMBERSHIP

ADDITIONS

(From December 5th, 1913, to January 8th, 1914.)

MEMBERS		Date of Membership.
BECK, HENRY PHILLIPS. Member of Administrative Board, City Hall, Richmond, Va.		Dec. 3, 1913
BOWLER, FRANK COLBURN. Chf. Engr., Great Northern Paper Co., Millinocket, Me.	Assoc. M. M.	Oct. 5, 1904 Dec. 31, 1913
BUGBEE, ALVIN. Supt., Trenton Water-Works, 565 Rutherford Ave., Trenton, N. J.		Dec. 3, 1913
COLLINS, EMMETT FILMORE. Asst. Engr., St. L. & S. F. R. R., 613 Frisco Bldg., St. Louis, Mo.		Dec. 3, 1913
COREY, RAY HOWARD. Gen. Mgr., Coos Bay Water Co., Marshfield, Ore.	Assoc. M. M.	Oct. 3, 1906 Dec. 3, 1913
CREAGER, WILLIAM PITCHER. Asst. Hydr. Engr., J. G. White Eng. Corporation, 43 Exchange Pl., New York City.		Oct. 1, 1913
DOYING, WILLIAM ALBERT EDWARD. Inspecting Engr., Isthmian Canal Comm. (Res., 3525 Fourteenth St., N. W.), Washington, D. C.	Assoc. M. M.	Feb. 1, 1910 Dec. 3, 1913
HANSEN, PAUL. Chf. Engr., Illinois State Water Survey, Univ. of Illinois, Urbana, Ill.	Jun. Assoc. M. M.	Jan. 3, 1905 April 1, 1908 Dec. 3, 1913
HOBART, ALBERT CLAUDE. 51 Beach St., Westerly, R. I.		July 2, 1913
HORNE, HAROLD WELLINGTON. Div. Engr., Board of Water Commrs. of Hartford, Farmington, Conn.	Assoc. M. M.	Oct. 2, 1907 Dec. 3, 1913
JANNI, ALFREDO CARLO. Cons. Engr., 28 West 46th St., New York City.		Dec. 3, 1913
JENNINGS, JOHN EDWARD. Chf. Engr., Tower Dept., Milliken Bros. Inc. (Res., 215 Westminster Rd.), Brooklyn, N. Y.	Assoc. M. M.	Nov. 30, 1909 Dec. 3, 1913
KEEFE, DAVID ANDREW. Cons. and Inspecting Engr., 115 North St., Athens, Pa.		Dec. 3, 1913
LUND, SVERRE. Chf. Estimator and Designing Engr., Eastern Bridge & Structural Co., Worcester, Mass.		Dec. 3, 1913
MARTIN, CHARLES DEWEY. Hydr. and Irrig. Engr., 548 Twenty-first St., Merced, Cal.		Dec. 3, 1913
ROBERTSON, ALEXANDER KING. (The McAlpine Robertson Constr. Co.), 806 Metropolitan Bldg., Vancouver, B. C., Canada		Dec. 3, 1913
SMITH, GEORGE EDSON PHILIP. Irrig. Engr., Arizona Agri. Experiment Station, 1195 Speedway, Tucson, Ariz.	Jun. Assoc. M. M.	Feb. 3, 1903 Sept. 6, 1905 Dec. 3, 1913
SMOOT, LLOYD DUVALL. Chf. Engr. of Jacksonville, Engrs.' Bldg., Jacksonville, Fla.		July 2, 1913

MEMBERS (*Continued*)

		Date of Membership.
THOMSON, ALEXANDER, JR. Div. Engr., Board of Water Supply, Walden, N. Y.......	Jun. Assoc. M. M.	Dec. 4, 1900 Feb. 3, 1904 Dec. 3, 1913
TROUT, HARRY EDGAR. The Scottwood, Robinwood and Monroe Sts., Toledo, Ohio........................		Nov. 12, 1913
VAN LIEW, JOHN EDGAR. Chf. Engr., Des Moines Plant, Des Moines Bridge & Iron Works, Des Moines, Iowa........	Assoc. M. M.	July 1, 1908 Nov. 12, 1913
VON PHUL, WILLIAM. Member of Firm, Ford, Bacon & Davis, 921 Canal St., New Orleans, La............		Dec. 3, 1913
WICKERSHAM, JOHN HOUGH. Lancaster, Pa..	Jun. Assoc. M. M.	May 6, 1902 April 6, 1909 Dec. 3, 1913

ASSOCIATE MEMBERS

ALLAN, THOMAS JOHN. Chf. Engr.. The Realty Syndicate, Syndicate Bldg., Oakland, Cal....................		Nov. 12, 1913
ATWATER, HUNTINGTON CLARK. Asst. Engr., Alexander Potter, 191 Claremont Ave., New York City........		Dec. 31, 1913
AYRES, LOUIS EVANS. Office Engr. for Gardner S. Williams, Cornwell Bldg., Ann Arbor, Mich.........................	Jun. Assoc. M.	Dec. 1, 1908 Dec. 3, 1913
BEE, CHARLES EVERETT. Div. Engr., Southern Alberta Land Co., Champion, Alberta, Canada............		Dec. 3, 1913
BELL, GEORGE EDWARD. Western Mgr., Dominion Bridge Co., Ltd., Winnipeg, Man., Canada...............		Nov. 12, 1913
BIGELOW, WILLIAM WALTER. Care, Sawyer & Moulton, 120 Exchange St., Portland, Me.................................	Jun. Assoc. M.	Feb. 1, 1910 Dec. 3, 1913
BORDEN, GUY. 130 West 71st St., New York City.........		Nov. 12, 1913
BOYD, WALTER LACY. Engr. in Chg. of Constr., State Phosphate Co., Bartow, Fla............................		Dec. 31, 1913
BRIGGS, EALY GRANNIS. Prin. Asst. Engr., Western Land Securities Co., Seney, Mich.....................		Nov. 12, 1913
BROOKS, DAVID WALKER. Mgr., Specialty Dept., Concrete Steel Co., 32 Broadway (Res., 112 West 72d St.), New York City............................	Assoc. Assoc. M.	Dec. 5, 1906 Dec. 31, 1913
BROOKS, JOSIAH RICHARDSON. Care, A. B. Brooks, 421 Peoples Gas Bldg., Chicago, Ill.................................	Jun. Assoc. M.	Feb. 2, 1909 Dec. 3, 1913
BUNKER, GEORGE HITCHELL. James Sherman & Sons, Foot of 26th St., Brooklyn, N. Y.....................		July 2, 1913
BURNELL, EUGENE. Care, Phœnix Constr. Co., Camp No. 3, Alexander, Idaho............................		June 4, 1913

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
CHESLEY, FRANK EPHRAIM. Res. Engr., Sanitary Sewer System, Beaver Falls, Pa.		Dec. 31, 1913
CHRISTIANSEN, EUGENE OLAF. Asst. Engr., U. S. Geological Survey, Wailuku, Maui, Hawaii		Nov. 12, 1913
COLE, EMMERT LUTHER. Structural Designer, Carnegie Steel Co., Baltimore, Md		Dec. 31, 1913
CUTLER, LEON GEORGE. With J. G. White & Co., 43 Exchange Pl., New York City (Res., 125 South Parkway, East Orange, N. J.)	Jun. Assoc. M.	April 5, 1910 Dec. 31, 1913
DEAN, DANIEL ABRAM. Supt. of Constr., James Stewart & Co., Care, New Winthrop Hotel, Winthrop Beach, Mass		Dec. 31, 1913
DU MOULIN, WALTER LOUIS. Supt., The Morenci Water Co., Morenci, Ariz	Jun. Assoc. M.	Feb. 1, 1910 Dec. 3, 1913
ENGLAND, ROLLO GUY. Civ. and Hydr. Engr., 117 Second St., Jackson, Mich		Nov. 12, 1913
FEIGEL, JOHN HENRY. Engr. for H. Osgood Holland, 262 Orange St., Buffalo, N. Y.	Jun. Assoc. M.	Oct. 6, 1908 Oct. 1, 1913
HALL, WARD. Asst. Engr., N. W. Pac. R. R., Irma, Cal		Dec. 3, 1913
HENDRICKS, SEWARD DANIEL. Engr. on Constr., Empire Eng. Corporation, 700 Telephone Bldg., Buffalo, N. Y.		Dec. 31, 1913
HUDMAN, ELLIS. Rock Springs, Wyo		Nov. 12, 1913
JENRICK, WILLIAM FREDERICK. Statistical Engr., The Foundation Co., 657 Bryant Ave., New York City		Dec. 3, 1913
KELSEY, LOUIS DE COU. City Engr., Aberdeen, Wash		Nov. 12, 1913
KITTS, JOSEPH ARTHUR. Corozal, Canal Zone, Panama		Dec. 3, 1913
MINER, ERWIN JOHN. Asst. Engr., Dept. of Public Service, Cincinnati, Ohio		Dec. 3, 1913
MORSE, HAROLD MARSTON. Designing and Const. Engr., H. Whitford Jones & Co., 2200 East 89th St., Cleveland, Ohio		Dec. 3, 1913
PAYROW, HARRY GORDON. Res. Engr., Lynn Additional Water Supply, 150 Bellevue Rd., Lynn, Mass	Jun. Assoc. M.	Feb. 4, 1908 Dec. 3, 1913
REA, RICHARD WILLIS. Engr., Cascade Irrig. Dist., P. O. Box 416, Ellensburg, Wash		Dec. 3, 1913
ROOT, JOSEPH EUGENE. Office Engr., Dept. of Public Works, 3436 Lyleburn Pl., Cincinnati, Ohio		Dec. 3, 1913
RUSSELL, CLAUD. Acting Div. Engr., Bureau of Public Works, Philippine Islands, Cebu, Cebu, Philippine Islands		July 2, 1913
SARGENT, EDWARD HAYNES. Asst. Civ. Engr., New York State Conservation Comm., 25 Delaware Terrace, Albany, N. Y.		Dec. 31, 1913

ASSOCIATE MEMBERS (*Continued*) — Date of Membership.

SKINNER, BENJAMIN BAKER. 97 Kenmore Pl., Brooklyn, N. Y. .... Nov. 12, 1913

SMITH, CHESTER KITCH. Bridge Engr., Spokane, Portland & Seattle Ry., Care, Chf. Engr.'s Office, Portland, Ore. Dec. 3, 1913

SNYDER, HUBERT EARL. Engr. of Constr., Logan Min. Co., Logan, W. Va. .... Jun. May 2, 1911; Assoc. M. Dec. 31, 1913

STAFFORD, EDWARD SATTLEY. Care, Casa Grand Val. Water Users' Assoc., Florence, Ariz. .... Nov. 12, 1913

STEESE, JAMES GORDON. Capt., Corps of Engrs., U. S. A., West Point, N. Y. .... Jun. Aug. 31, 1909; Assoc. M. Dec. 3, 1913

WARNER, JAMES MADISON. Chf. Engr., Onondaga Litholite Co., 301 Slocum Ave., Syracuse, N. Y. .... Jun. April 6, 1909; Assoc. M. Dec. 3, 1913

WEST, WADE CLARENCE. Care, Bureau of Public Works, Manila, Philippine Islands .... Sept. 3, 1913

WILKINS, HOMER JENNER. Drainage Engr., 1819 West 9th St., Oklahoma, Okla. .... Jun. Oct. 31, 1911; Assoc. M. Dec. 3, 1913

ASSOCIATES

WILSON, HUGH MONROE. Vice-Pres. and Gen. Mgr., McGraw Pub. Co., Inc., 239 West 39th St. (Res., 375 Park Ave.), New York City .... Dec. 3, 1913

JUNIORS

ALLEN, FRANKLIN REA. City Engr., Pine Bluff, Ark. .... Dec. 3, 1913

ANGAS, ROBERT MOORE. Engr. in Chg., Development Work, Indian River Assoc., Hobe Sound, Fla. .... Dec. 3, 1913

BAKER, DONALD MCCORD. Asst. Engr., U. S. Indian Irrig. Service, Box 445, Albuquerque, N. Mex. .... Dec. 3, 1913

BARBER, NORMAN NATHANIEL. Asst. Engr., C., M. & St. P. Ry., Care, C., M. & St. P. Junction, Ottumwa, Iowa (Res., 6202 Ingleside Ave., Chicago, Ill.) .... Dec. 3, 1913

BECK, RALPH ERNEST. 35 Conklin Ave., Newark, N. J. .... Dec. 3, 1913

BITHER, TOM ALLEN. With City Engr., 1630 Josephine St., Berkeley, Cal. .... Oct. 1, 1913

BLEY, CHARLES NICHOLAS. 2107 Hearst Ave., Berkeley, Cal. Nov. 12, 1913

BRINGHURST, HORACE MORTON. Engr. Draftsman, Bldg. Dept., City of Seattle, 552 Eighteenth Ave., Seattle, Wash. .... Nov. 12, 1913

BRINKERHOFF, GEORGE LOCKWOOD. Care, Isthmian Canal Comm., Gatun, Canal Zone, Panama .... Oct. 1, 1913

CLAUSEN, STANLEY JAMES. With Mississippi River Comm., 1311 Liggett Bldg., St. Louis, Mo. .... July 2, 1913

CRANE, JACOB LESLIE, JR. 823 Scarritt Bldg., Kansas City, Mo. .... Dec. 3, 1913

JUNIORS (*Continued*)	Date of Membership.
DURYEA, ROBERT FRANCIS. 1280 California St., San Francisco, Cal	Oct. 1, 1913
FLOOK, LYMAN RUSSELL. Asst. to Supt. of Bldgs. and Grounds Dept., Univ. of Michigan, 616 Church St., Ann Arbor, Mich	Dec. 3, 1913
FRANK, LESLIE CARL. 1317 Madison Ave., Baltimore, Md	Dec. 3, 1913
GOWEN, JOHN FELLOWS. Asst. Engr., Board of Water Supply, City of New York, Ossining, N. Y	Oct. 1, 1913
HABERLE, EDWARD LOUIS. 2425 Lyndale Ave., North, Minneapolis, Minn	Dec. 3, 1913
HENNING, CHARLES SUMNER, JR. P. O. Box 298, Vale, Ore	Dec. 3, 1913
HINDE, CHARLES. City Engr.'s Office, Vancouver, B. C., Canada	Nov. 12, 1913
HUXTABLE, WILLIAM GUIREY. With W. E. Ayres, Cons. Engr., Earle, Ark	Nov. 12, 1913
KAMINSKY, BENNETT. Chf. Draftsman, Office Engr., M. of W., The Indianapolis Union Ry., Room 4, Union Station, Indianapolis, Ind	July 2, 1913
KIRCHGRABER, HAMLIN EARLE. 801 West End Ave., New York City	Dec. 3, 1913
McGEE, HAROLD GILBERT. 303 Hartman Bldg., Columbus, Ohio	Dec. 3, 1913
MILLER, JOHN OWEN. 1595 Clay St., San Francisco, Cal	July 2, 1913
OCKERT, FREDERICK WILLIAM. 254 West 104th St., New York City	Dec. 31, 1913
OSGOOD, MANLEY. City Engr., Ann Arbor, Mich	Dec. 31, 1913
ROBSON, RALPH EWART. Templeton, Cal	Oct. 3, 1911
SEGURA, VALERIANO. Asst. Engr., Bureau of Public Works, Cebu, Cebu, Philippine Islands	Oct. 1, 1913
STEVENS, HAROLD CROSBY. Prin. Asst. Engr., Blair & Drave, 1205 Commercial Bldg., Charlotte, N. C	Dec. 31, 1913
WALKER, GUY BURT. City Engr.'s Office, City Hall, Wilkes-Barre, Pa	Dec. 3, 1913
WENDELBOE, LEE. Care, U. S. Reclamation Service, Logan, Utah	Dec. 3, 1913
WOODRUFF, GLENN BARTON. Bridge Designer with L. V. R. R., 438 Cherokee St., South Bethlehem, Pa	Dec. 3, 1913

RESIGNATIONS

MEMBERS	Date of Resignation.
BROWN, ROBERT CALVIN	Dec. 31, 1913
FENN, ROBERT WILLSON	Dec. 31, 1913
GREENE, FRANCIS VINTON	Dec. 31, 1913

MEMBERS (*Continued*)	Date of Resignation.
Hayt, Stephen Thurston, Jr.	Dec. 31, 1913
Jackson, Thomas Herbert	Dec. 31, 1913
Kauffmann, William Frederick	Dec. 31, 1913
Kenrick, Robert Boteler	Dec. 31, 1913
Martin, Kingsley Leverich	Dec. 31, 1913
von Emperger, Fritz Edler	Dec. 31, 1913
ASSOCIATE MEMBERS	
Alexander, John Howard	Dec. 31, 1913
Cross, John Halsey	Dec. 31, 1913
Forrest, George Munro	Dec. 3, 1913
Kreiner, Harry Peter	Dec. 31, 1913
McCoy, Charles Ephraim	Dec. 31, 1913
Meyers, Clarence William	Dec. 31, 1913
Morrison, Charles Edward	Dec. 31, 1913
Shortt, John Haggerty	Dec. 31, 1913
ASSOCIATES	
Drummond, Thomas Joseph	Dec. 31, 1913
McBurney, Henry	Dec. 31, 1913
Meyer, Henry Coddington, Jr.	Dec. 31, 1913
Scribner, Gilbert Hilton, Jr.	Dec. 31, 1913
JUNIORS	
Calder, John Webster	Dec. 31, 1913
Gundlach, George Christian	Dec. 31, 1913
Hinrichs, Adolf	Dec. 31, 1913
Holloway, Arthur Power	Dec. 31, 1913
Kincaid, Murtland	Dec. 31, 1913
Leete, Robert Burt	Dec. 31, 1913
MacLeish, Gordon Grant	Dec. 31, 1913
Nagel, Theodore	Dec. 31, 1913
Robinson, Ward Reid	Dec. 31, 1913
Whitman, William Satterwhite	Dec. 31, 1913

DEATHS

Clarkson, Robert Cooke. Elected Junior, January 5th, 1887; Member, January 2d, 1901; died December 26th, 1913.

Clayton, Henry Helm. Elected Junior, December 3d, 1907; died in April, 1913.

Compton, Alfred G. Elected Associate, September 5th, 1877; died December 13th, 1913.

Cotton, Joseph P. Elected Member, June 7th, 1876; died December 27th, 1913.

Forbes, Murray. Elected Associate Member, June 5th, 1907; died December 28th, 1913.

GRAY, HARRY WOY. Elected Associate Member, November 1st, 1910; died December 14th, 1913.

LABELLE, HENRY FRANCIS. Elected Member, April 6th, 1898; died December 12th, 1913.

LUTZ, ULYSSES STANISLAUS. Elected Member, March 5th, 1912; died December 8th, 1913.

MACCRACKEN, GEORGE GERE. Elected Junior, February 5th, 1901; Associate Member, February 3d, 1904; died August 1st, 1913.

MILES, JOHN WILEY. Elected Associate Member, October 2d, 1901; Member, April 6th, 1909; died in January, 1912.

RICHMOND, HENRY A. Elected Fellow, July 7th, 1870; died May 10th, 1913.

VLIEGENTHART, JOHANNES CORNELIS. Elected Associate Member, June 5th, 1907; died in October, 1913.

WOOD, HENRY SHOTWELL. Elected Member, May 1st, 1907; died December 5th, 1913.

Total Membership of the Society, January 8th, 1914, 7 284.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(December 2d, 1913, to January 5th, 1914)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.

(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenicurwesen*, Wiesbaden, Germany.
(82) *Mining and Engineering World*, Chicago, Ill., 10c.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.

LIST OF ARTICLES

Bridges.

Trusses without Diagonals in Reinforced Concrete.* V. J. Elmont. (5) Vol. 27, Pt. 1.
The Erection of the Municipal Bridge, St. Louis, Mo. W. H. Radcliffe. (4) Nov.
Vertical Lift Drawbridges.* (12) Serial beginning Nov. 28.
Proportioning of Long-Span Truss and Cantilever Bridges. Joseph Mayer. (3) Dec.
American River Bridge.* A. M. Wolf. (87) Dec.
Influence of the Position of Crown Hinge on the Weight and Deflection of a Three-Hinged Spandrel-Braced Arch.* M. A. Beltaire, Jr., and R. W. Parkhurst. (36) Dec.
Hard Roads and Permanent Bridges.* Daniel B. Luten. (67) Dec.
Lifting the One Hundred and Thirty Million Pound Quebec Bridge.* H. F. Stratton. (9) Dec.

*Illustrated.

Bridges—(Continued).

Transferring Bridge Spans on Barges. (13) Dec. 4.
Erecting the St. Lawrence River Bridge.* (23) Dec. 5.
Unusual Abutment Design, Hollow Reinforced-Concrete Triangular Boxes at Three-Level Crossing of Creek, Railroad Avenue at Montclair.* (14) Dec. 6.
Viaduct Floor of Hollow Concrete Tile.* (14) Dec. 6.
Observations on Bridge Expansion.* H. A. Loser. (13) Dec. 11.
The Sydney Harbour Bridge.* (11) Dec. 12.
Safeguarding Displaced Viaduct Piers.* (14) Dec. 13.
Moving Loads on Steel Bridges. (14) Dec. 13.
Bridge Construction at Newcastle, N. B.* C. A. Wentworth. (96) Dec. 18.
Pont Notre-Dame Reconstruction, Paris; Arch Replacement under Traffic.* (13) Dec. 18.
Schuylkill River Bridge Improvements.* (15) Dec. 19.
The Lower Ganges Bridge.* (23) Dec. 19.
Largest Bascule Bridge, Double-Track, Single-Leaf Span for Baltimore & Ohio Bridge over Calumet River, Chicago, has Length of 230 Feet.* (14) Dec. 20.
Deep-Water Piers for Passaic and Hackensack River Bridges, Building Pile Foundations and Depositing Concrete on Them Under Water in Cofferdams and in Submerged Forms.* (14) Dec. 20.
Bridges, Stations and Masonry Work on the Montclair Branch Improvements, Delaware, Lackawanna and Western.* (14) Dec. 20.
Substructure Work for Canadian Pacific Railway Bridge at Mud Lake, Ont.* (86) Dec. 24.
Cost of Dismantling an Old Highway Bridge and Erecting New Truss and Girder Spans. George Harper. (86) Dec. 24.
Two Large Concrete Arches on a Swiss Mountain Railway.* (13) Dec. 25.
High Cantilever Bridge in Mexico.* (15) Dec. 26.
Preliminary Considerations in the Design of Opening Bridges. Henry Grattan Tyrrell. (15) Dec. 26.
Dreigelenkbogen aus Beton mit grosser Spannweite und kleiner Konstruktionshöhe. D. Kutschke. (51) Serial beginning Sup. No. 23.
Der Neubau der Arndt-Strassen-Ueberführung in Königsberg i. Pr.* Ernst Schönwald. (51) Sup. No. 23.
Die Erschütterungen bei den Sprengungen der Strompfeiler der alten Kölner Eisenbahnbrücke.* Christfreund. (40) Nov. 8.
Das Segmentschütz der Freiarche in Spandau.* Klehmet. (40) Nov. 26.
Die Dritte Phase der gebogenen Eisenbetonträger.* Max R. v. Thullie. (53) Nov. 28.
Breitflanschige Träger. Schaper. (40) Nov. 29.
Hängebrücke über die Luzège in der Kleinbahn von Ussel nach Tulle. Landsberg. (40) Dec. 10.
Eisenbetoncaisson der Elbebrücke in Obristvi (Böhmen).* Lumír Kapsa. (78) Dec. 15.
Strassenbrücke mit aufgehängter Fahrbahn.* C. Ritter. (78) Dec. 15.
Kontinuierliche Bogenträger auf elastischen Stützen.* A. Ostenfeld. (78) Dec. 15.

Electrical.

Report of Committee on Street Lighting, American Society of Municipal Improvements.* (99) 1912.
Phase Compensation.* G. H. Eardley-Wilmot. (73) Nov. 28.
A 25 000-kw. Parsons Turbo-Alternator.* (26) Nov. 28.
Operation of Transmission Lines.* Lee Hagood. (42) Dec.
The Dielectric Strength of Thin Insulating Materials. F. M. Farmer. (42) Dec.
High-Voltage Engineering.* F. W. Peek, Jr. (3) Dec.
Street Lighting with Tungsten Lamps.* (60) Dec.
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Specifications for Brick Pavements.* Committee on Standard Specifications, Am. Soc. of Mun. Improvements. (99) 1912.
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Specifications for Concrete Pavements, Cement Sidewalks, and Concrete Curb and Gutter. Sub-Committee on Concrete Paving, Am. Soc. of Mun. Improvements. (99) 1912.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XL. JANUARY, 1914. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

GROUTED CUT-OFF FOR THE ESTACADA DAM.

By Harold A. Rands, Assoc. M. Am. Soc. C. E.

To be Presented February 18th, 1914.

Early in 1912, the Portland Railway, Light and Power Company, which is the principal public service corporation of Portland, Ore., completed what is known as the Estacada Hydro-Electric Development. This work, which includes an Ambursen dam, 90 ft. high, with power-house and transmission line, has already been described in several technical journals, and to redescribe these features is not the object of this paper. Inasmuch, however, as the treatment of the foundation material by the grouting process is the greatest thus far attempted, it is believed that a setting forth of this particular feature of the work will prove of interest to many engineers.

The location of the Estacada or River Mill plant, as it is sometimes called, is some 25 miles southeast from Portland, in the extreme western foot-hills of the Cascade Range, at the point where the Clackamas River, issuing from its long and tortuous canyon, begins its course through a rolling alluvial plain. The water-shed above the dam is approximately 800 sq. miles, and though the minimum flow for a short time in the dry months of August and September may fall to 700

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions.*

sec-ft., the average, of course, is much higher, and at times of extreme flood, like that of November, 1909, may rise to 40 000 sec-ft.

Previous Experience.—The Company was not without experience in construction work in these regions. For years it had operated a hydro-electric plant at Oregon City, where the Willamette River drops over a basaltic ledge with a fall of 40 ft. In 1906 it completed the Cazadero Station, 3 miles above the new Estacada plant. The operating head at this station is 130 ft., but the dam, which is $1\frac{3}{4}$ miles up the river from the plant, raises the water only 45 ft., the remainder being obtained by the flume, ditch, and dike by which the water is conveyed to the forebay. This dam is of logs, and though there is some seepage passing through and around it, this never has been excessive, and Mr. T. W. Sullivan, Hydraulic Engineer for the Company, who completed the work after it had been acquired in an incompleted state from other interests, states that no springs have at any time appeared, either in the bed of the river or in the vertical sides of the gorge, as a result of the dam.

In 1907 the Company began work preliminary to the construction of a dam about 150 ft. high at the head of slack water 2 miles above the Cazadero Dam. Here, under the supervision of S. C. Hulse, Assoc. M. Am. Soc. C. E., extensive borings were put down and tests of various kinds were made to determine the feasibility of erecting so high a structure on the formation found. These borings disclosed rock of so porous a nature that even a moderate pressure applied to the casings of the drill holes produced a leakage of many gallons per minute. Pressure applied to one hole would also cause the water to rise in other holes some distance away. The engineers had the benefit of the knowledge gained by the above construction and experience when in 1909 the Estacada site became the property of the Company.

Preliminary Borings.—At the site proper the river flows through a gorge of lava conglomerate, the sides of which are nearly vertical and from 50 to 70 ft. high. They are capped with a sloping over-burden of gravel, small boulders, and clay from 2 to 20 ft. deep. Preliminary borings, ranging in depth from 60 to 250 ft. and aggregating 2 500 ft. of drilling, were put down on both sides of the river for a distance of several hundred feet up and down stream, as well as on the island which later became a part of the dam.

That the general condition of the Estacada site would be similar to that at the site of the proposed 150-ft. dam previously mentioned was to be anticipated by one having a general knowledge of this region, of its geology, and of the great lava flow which now covers it. Concerning this flow the following is quoted from the Encyclopedia Britannica:

"This is probably the grandest lava-flow known to geology, covering as it does an area of about 200 000 square miles. Commencing in middle California as separate streams, in northern California it becomes a flood, completely mantling the smaller and flowing around the greater inequalities. In northern Oregon and Washington it becomes an absolutely universal flood, beneath which the whole original face of the country, with its hills and dales, mountains and valleys, lies buried several thousand feet. It covers the greater portion of northern California and north-western Nevada, nearly the whole of Oregon, Washington, and Idaho, and runs far into British Columbia on the north. The average thickness is probably not far from 2 000 feet, and the greatest (shown where the Columbia, Des Chutes, Snake, Salmon, and other rivers cut through it) about 4 000 feet. To produce this many successive flows took place, and a very long period of time must have elapsed during which the volcanic actions were going on."

Notwithstanding the universality of this great flood, the formations along those streams flowing westward from the Cascade Range with a rapid descent to the sea, as a rule, present less secure banks than do those traversing the high undulating plateau to the east, as even a casual observer who is familiar with the Snake or Des Chutes River Canyons, with their rim rock and columnar-like structure, may have noted. Concerning this western slope formation Mr. J. S. Diller, of the U. S. Geological Survey, in a report made in 1909 covering the site of the proposed 150-ft., Clackamas River Dam, expressed himself as follows:

"The rocks of this canyon wall are of four forms; volcanic breccia, lava sheets, volcanic dikes and terrace gravels, the volcanic breccia being most abundant.

"The conditions that confront the engineer along the Clackamas river in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade range from the Columbia river in Oregon to the Feather river in California, one of the most important power belts in the United States, and the successful solution of the problem it presents at one point will greatly facilitate the work elsewhere."

To the above, and possibly of greater interest to the engineer, may be added a statement from one, who, from an engineering standpoint, has probably a first-hand knowledge of the entire region second to no one. Certainly, in terseness of language and clearness of expression it leaves nothing to be desired. John F. Stevens, M. Am. Soc. C. E., in a report on the Estacada Dam Site, expressed himself as follows:

"The site of the proposed works lies in the west slope of the Cascade range, and this range is well known to be, geologically taken, as of volcanic origin. Go where we may, for hundreds of miles North or South, we will find the upper valleys, canyons and river beds filled with what we call volcanic débris. Sometimes this volcanic matter appears as the hardest kind of basalt, sometimes as loose and friable as garden soil and generally occurs in every imaginable character and degree of solidity between the two, and it is not probable that nature has made any exception in this particular case. The limited number of test pits and borings which have been made show clearly that as far as the area they cover no well-defined, extremely hard masses of rock exist—that the rock is soft and shades away into clay, the latter appearing under superficial examination much like a half burned brick. The whole formation to the non-geologically scientific man seems to be a mass of soft conglomerate, and this description is borne out in striking manner by the appearance of the nearly vertical walls of the canyon at the site of the dam and immediately adjacent."

Fig. 1, showing the left channel and vertical face of the island in the natural state, gives a good idea of the appearance of the rock. The pitted and scarred surface of the weathered breccia, with its hard irregular fragments embedded in the weakly cemented matrix of indifferent sand, is well shown. In some holes the formation, as shown by the preliminary borings, seemed to be much the same down to the bottom, and in others ledges of hard rock, from 5 to 20 ft. thick, were penetrated; in some the sandy matrix, most common in surface croppings, would change with depth to an even-textured clay, sufficiently compact to give a fair percentage of core; in several holes, though not among those put down on the site finally selected for the dam, this clay at considerable depth deteriorated to such a consistency that for many feet the diamond bit was replaced with a star bit, with which, rotating as did the diamond bit, very rapid progress was made for many consecutive feet.

The sandy matrix and, to a lesser degree, the clay-like matrix were ground away by the attrition of the drills and the harder fragments of

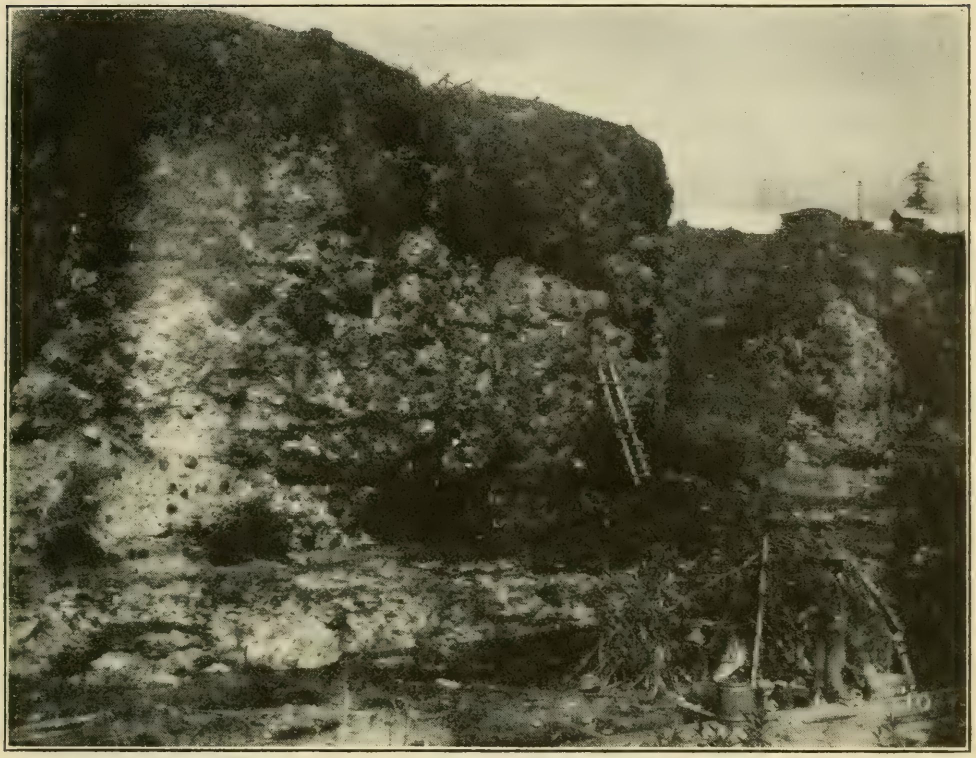


FIG. 1.—LEFT CHANNEL. WEATHERED BRECCIA IN NATURAL STATE.

FIG. 2.—DAM, WITH POWER-HOUSE NEARING COMPLETION.

rock so that an average for the diamond bit, using the single-tube core barrel for the Sullivan diamond drill, operated by that Company's expert runners, was slightly less than 50 per cent. The double-tube barrel as used on part of the work gave core amounting to slightly less than 80 per cent. In these borings fragments of wood were passed through at different depths, and in subsequent excavation several stumps, one about 30 in. in diameter, and portions of logs in part petrified and in part in a semi-decayed state were encountered, so that altogether the term "débris" seems most fittingly to describe the material.

The Island Location.—A question constantly being asked by visiting engineers during construction was: Why was the dam located at the island rather than in the narrow gorge some few hundred feet up stream? The reply to this query is as follows:

In the beginning, three sites were under consideration, Site A at the railroad bridge, Site B, 200 ft. below Site A, and Site C, at the island. These are shown on Fig. 3. The reasons which led to the selection of Site C seem to have been:

(1) The soft clay encountered in the preliminary borings already mentioned;

(2) The great length of spillway which was essential, as the crest of the new dam was to be at the same elevation as the low-water tail-race discharge of the Cazadero plant above;

(3) Ease in handling the water during construction.

At the time of making the decision it was believed that the dam in the right and left channels could with safety be made to abut against the island. Later, this was deemed to be taking chances which amounted to gambling with fate, so that over the up-stream side of the island, cut to the slope of the dam, an Ambursen deck, minus most of the steel, was extended. The down-stream slope was also carpeted for a considerable distance, so that as it stands the island is incorporated in the dam, and this portion of the structure is unique in being "a hollow dam" filled with island.

The plan for grouting the foundation originated with Messrs. Sellers and Rippey, of Philadelphia, Consulting Engineers on the work. Their conception of the problem and what they hoped to accomplish is well set forth in a paper by H. V. Schreiber, M. Am. Soc. C. E., and a member of the above firm. This paper was presented

at the Eighth Annual Convention of the National Association of Cement Users. Mr. Schreiber in part said:*

"After a careful study of the conditions disclosed by the investigation it was decided that in order to secure an impermeable foundation it was necessary to practically provide for actually changing the structural character of the underlying formation. To do this Mr. S. Howard Rippey, chief engineer of Messrs. Sellers and Rippey, engineers on the work, recommended that the most promising method would be the solidification of the porous material by the introduction of cement grout under pressure, but that the success of the work must be susceptible of demonstration by actual test before the superstructures should be started. Thorough inquiry failed to disclose any precedent for the use of grout for the general treatment of foundations, although it was found that cavities in limestone rock under the New Croton Dam had been filled with grout, much as a dentist would fill a cavity in a tooth. The grouting method had also been used in filling back of lining walls in tunnels, etc. Notwithstanding the absence of precedent, it was decided to proceed with a grouting scheme and a program was outlined for preventing leakage of water from the reservoir created by the dam, under or around the dam, to the low tail water level below the dam and the experiments which should be made to properly demonstrate the efficacy of the method before the complete development should be undertaken were prescribed. The general idea provided for drilling a double line of holes of an average depth of 50 ft. under the heel of the dam across the entire valley to and under the shore abutments and the subsequent forcing into each of these holes of grout of such consistency as to percolate through the entire substructure.

"It was recognized that experiments would be necessary to determine the proper spacing of the grout holes and their depth, which would insure sufficient diffusion of the grout through the varying material encountered to create a continuous impermeable barrier to prevent seepage of water from the reservoir under the hydrostatic head which would be created by the dam. After drilling the double line of holes, the program contemplated the test of each hole with water pressure, a record being kept of the quantity escaping and the pressure applied to each hole. Upon the completion of the tests, the cement grout was to be pumped into the holes under pressure and after allowing time for hardening, a third line of holes was to be drilled midway between the first two or outer lines and tested with water pressure. The idea was that if water pressure applied to the centre holes at or slightly above the hydrostatic pressure to which the rock would be subjected by the water in the reservoir after completion of the dam and no appreciable leakage occurred, it would be rea-

* *Engineering Record*, March 23d, 1912, p. 314.

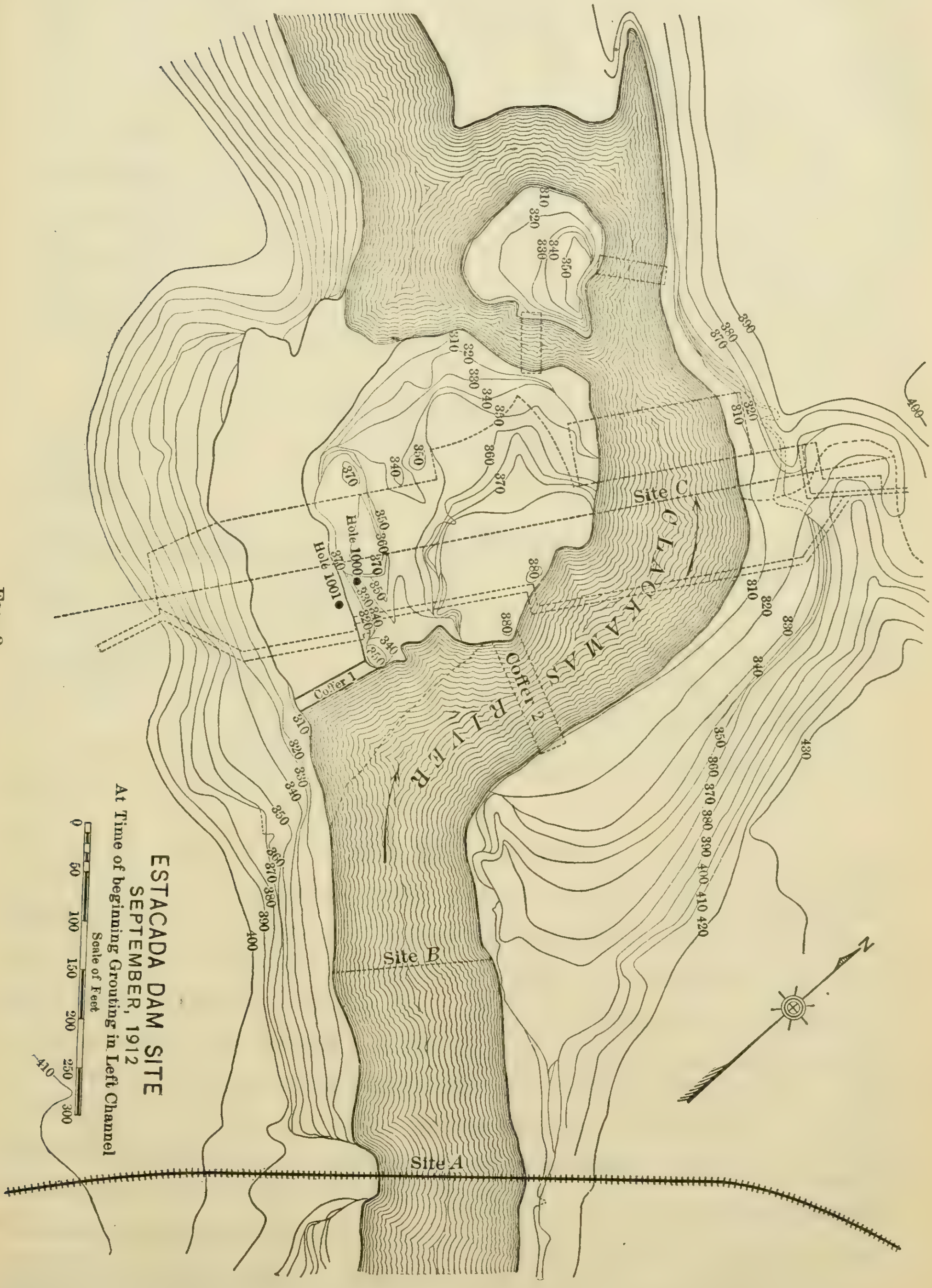

ESTACADA DAM SITE
SEPTEMBER, 1912
At Time of beginning Grouting in Left Channel
0 50 100 150 200 250 300
Scale of Feet
CLACKAMAS RIVER
Site A
Site B
Site C
Coffer 1
Coffer 2
Hole 1000
Hole 1001

FIG. 3.

sonably certain that the cement grout had proven effective in making the entire foundation impermeable."

The drilling for the grouting included 555 holes, aggregating 34 038 lin. ft. For a short time the Sullivan diamond drills, operated by steam, and also the Davis-Calyx shot drills, operated by electric power, were used. The diamond drills, being required at the site of the proposed 150-ft. dam, were released after putting down 1 119 lin. ft., and from that time to the end the shot drills were used exclusively. These drills are made by the Ingersoll-Rand Company, and those used on this work, with one exception, were the small "Class G-O" machine. These may be operated by hand if speed is of secondary consideration, but on this work each was driven by a 5-h.p. direct-current motor which

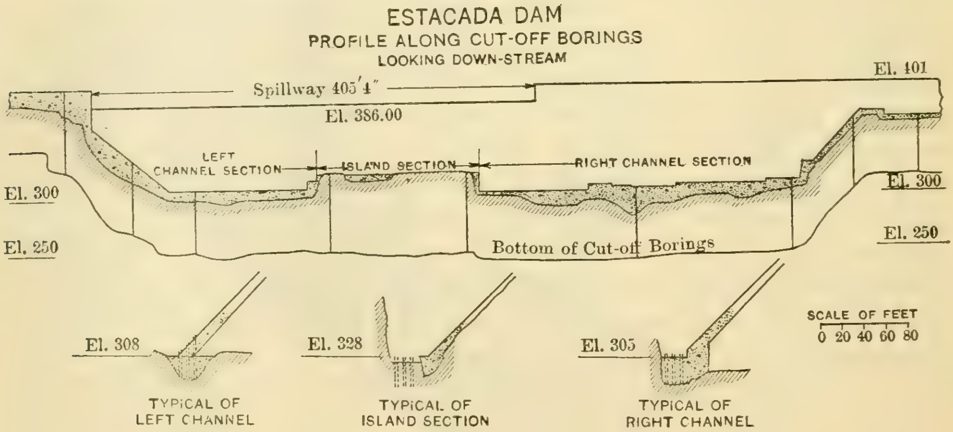


FIG. 4.

gave the bit a speed of 187 rev. per min. The bit had an outside diameter of 2⅜ in. and gave a 1½-in. core. Seven of these drills were purchased, but only for a short time were all working; the average for the 32 919 ft. put down by them was 13 ft. per 10-hour shift.

Grouting.—The grouting was done with a Canniff air-stirring grouting machine, made by the Tide Water Iron Works, Hoboken, N. J. This wonderfully complete and self-contained machine, in which the air does the mixing and discharging with very little outside work, having been used in the grouting of tunnels and similar work, is now too well known to require description here.* As used on the Estacada Dam, air was supplied at a pressure of 250 lb. per sq. in., and in

* One of these machines was sold later to the U. S. Reclamation Service, and used in grouting the Lahontan Dam of the Truckee-Carson Project. An excellent description of the machine, with drawings, will be found in an article by D. W. Cole, M. Am. Soc. C. E., in *Engineering News*, April 3d, 1913, p. 647.

grouting it was customary to start each hole at a pressure of 25 lb., or whatever was required to start the discharge, and as the hole tightened to increase the pressure, finally ending when at 200 lb. no more grout could be forced in. On this work only neat cement was used. The mixture varied from 1 part cement and 1½ parts water, by volume, to 1 part cement and 15 parts water, the best results seeming to be with 1 part cement and 3, 4, or 5 parts water. In order to prevent the choking of the holes, the scheme was tried of discharging the grout at the bottom of the hole through an inner pipe passing through the cap which sealed the top of the main casing. This method, after a thorough trial, was found of no benefit, and was discontinued.

Arrangement of the Holes.—The plan for the grouting, as proposed by W. L. Fitzgerald, Assoc. M. Am. Soc. C. E., Designing Engineer for the firm of Sellers and Rippey, was that of two lines of holes, 6 ft. apart with 6 ft. between holes in the same line.

These were known as primary holes. To test the effect of the grouting of these primary holes it was proposed that, after allowing a reasonable time for the cement in the primary holes to set, a proving hole should be put down in the center of each square formed by the four primary holes. If this tested tight or reasonably so, that square would be complete. If it did not, additional proving holes would be put down, first on one side and then on the other of the first, so that if all were required in the end there would be four primary and three proving holes in each group.

The Left Channel.

The first grouting done was in the left channel in September, 1910. Across this channel a low coffer-dam, designated on Fig. 3 as Coffer-dam No. 1, had been placed. Back of this a cut-off trench from 8 to 9 ft. deep had been excavated, and through 3-in. casings set with cement 4 ft. in the bottom of this trench, holes were drilled 46 ft. or to a total depth of 50 ft. from the bottom of the cut-off.

The head against the coffer-dam was seldom more than 2 ft., and at times of minimum load at the Cazadero plant, the water receded to the deep-water channel indicated on Fig. 3. Insignificant as this head was, so porous was the ground that it sufficed to cause many of the casings to flow continuously as Artesian wells. Conversely, when pressure was applied to the casings of a number of the holes, leakage

was observed at several points in the river bed, which here was conglomerate bare of gravel, nearly out to the deep-water channel.

Unfortunately, for purposes of comparison, these first holes were tested by a pump taking water from a tank provided with a gauge reading in gallons direct. The pressure gauge was attached to the elbow connecting the water line to the casing of the hole, and the pressures as registered by this and the water taken from the tank were recorded at intervals of 1 min. Tests were made at one-half depth and at full depth for each hole. This method of testing was found to be unsatisfactory, and subsequent tests were made by gravity pressure from an elevated tank also provided with a gauge reading in gallons.

The first attempt to grout was made with the bottom of the cut-off trench bare, but so great was the loss of pressure and the escape of grout through seams in the rock that operations were suspended until the bottom had been protected with a mat of concrete from 2 to 3 ft. thick. At the time of beginning the grouting, though the drills were still working at that point, there had been put down some twenty testing or primary holes. So free was the communication that in grouting one hole it was necessary to cap the neighboring ungrouted holes for some distance in each direction, though it was customary to leave the caps of these holes slightly loose until the air and clear water first forced up gave place to the grout.

Twenty-five holes grouted at this time show results as follows: Average pressure, 19.4 lb.; average leakage, 85.7 gal. per min.; total cement, 167.25 bbl.; average per hole, 6.69 bbl.; most cement in any hole, 40 bbl.

As soon as the grouting was done across the bottom of the left channel, the urgency of getting to work on the power-house and other construction in the right channel made it necessary to blow up the coffer-dam, thus for a time ending operations at this point. This too was unfortunate, for, though it was not known at the time, operations had been commenced in the worst and most porous material encountered on the entire job, and, as future developments were to show, it had to be left but slightly better, apparently, for these efforts.

The Right Channel.

That the construction of the dam proper might proceed with the least possible hindrance from the drills, William H. Cushman, M.

PRIMARY HOLES GROUTED BEFORE CONCRETING

Hole No.	711	712	713	714	715	716	717	718	719	720	
Test Date	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/6'11	4/6'11	4/6'11	
Gall's per Min.	41	100	76	78	96	47	40	75	40	15	
Grout Date	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/10'11	4/21'11	4/24'11	
Bbls. Cement	7.75	5	1.25	25	6.25	6.25	9.25	2.5	1.25	10	
Hole No.	721	722	723	724	725	726	727	728	729	730	Average
Test Date	4/6'11	3/18'11	3/18'11	3/18'11	No	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	
Gall's per Min.	35	75	7	94	Test	4	1	15	30	9	46.2
Grout Date	4/21'11	4/11'11	4/10'11	4/21'11	4/12'11	3/19'11	4/10'11	4/10'11	3/20'11	3/20'11	
Bbls. Cement	3.25	1	3	2	3.75	1	2.75	3.5	1.75	6.25	5.14
Hole No.	511	512	513	514	515	516	517	518	519	520	
Test Date	4/21'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/20'11	4/6'11	4/6'11	4/6'11	
Gall's per Min.	80	81	90	69	95	40	48	55	79	20	
Grout Date	4/30'11	4/30'11	4/30'11	4/30'11	4/30'11	4/26'11	4/30'11	4/10'11	4/17'11	4/24'11	
Bbls. Cement	4.25	1	1	1	5	2.25	4	3.75	4.5	1.75	
Hole No.	521	522	523	524	525	526	527	528	529	530	Average
Test Date	4/6'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	3/18'11	
Gall's per Min.	40	91	93	34	26	93	80	83	13	5	60.75
Grout Date	4/24'11	3/21'11	4/21'11	4/21'11	3/19'11	4/12'11	3/20'11	3/20'11	3/19'11	3/19'11	
Bbls. Cement	5.5	6.5	2	2.5	1.25	3	1	2	3.75	1	2.85

INTERMEDIATE PRIMARY HOLES GROUTED AFTER CONCRETING

Hole No.	711*a*	712*a*	714*a*	715*a*	718*a*	719*a*	720*a*	721*a*	722*a*	
Test Date	6/7'11	6/9'11	6/13'11	6/9'11	6/15'11	6/21'11	6/16'11	6/15'11	6/20'11	
Gall's per Min	22	2	2.5	2	1	2.7	20	40	15	
Grout Date	6/7'11	6/9'11	6/13'11	6/9'11	6/15'11	6/23'11	6/17'11	6/15'11	6/20'11	
Bbls. Cement	11.25	1	1	16.75	1.5	1.25	1	1.75	1.25	
Hole No.	724*a*	725*a*	726*a*	727*a*	527*a*	728*a*	528*a*	729*a*	529*a*	Average
Test Date	6/16'11	6/21'11	6/28'11	6/16'11	7/2'11	6/21'11	7/6'11	6/17'11	6/28'11	
Gall's per Min	2.7	1.2	1.8	3.5	2.5	4.5	1	63	46	13.0
Grout Date	6/17'11	6/23'11	6/28'11	6/17'11	7/2'11	6/23'11	7/6'11	6/23'11	7/2'11	
Bbls. Cement	1.5	1	1.25	1.25	1	1	1.25	1	3	2.72

PROVING HOLES

Hole No.	911*b*	913*b*	915*b*	916*b*	918*b*	919*b*	921*b*	923*b*	925*b*	927*b*	928*b*	929*b*	Average
Test Date	6/25'11	6/9'11	6/7'11	8/16'11	7/17'11	6/20'11	6/25'11	6/26'11	7/5'11	7/8'11	7/13'11	7/8'11	
Gall's per Min	2.5	2	4.2	2.1	1.3	14.6	4	3.5	1.2	3	12.5	44.5	7.95
Grout Date	8/12'11	8/12'11	8/12'11	8/17'11	8/12'11	8/12'11	8/1'11	7/14'11	7/14'11	7/13'11	7/13'11	7/13'11	
Bbls. Cement	4.5	0.75	4	1.25	0.75	1.75	1.50	1	1.25	1.25	1.25	1.5	1.73

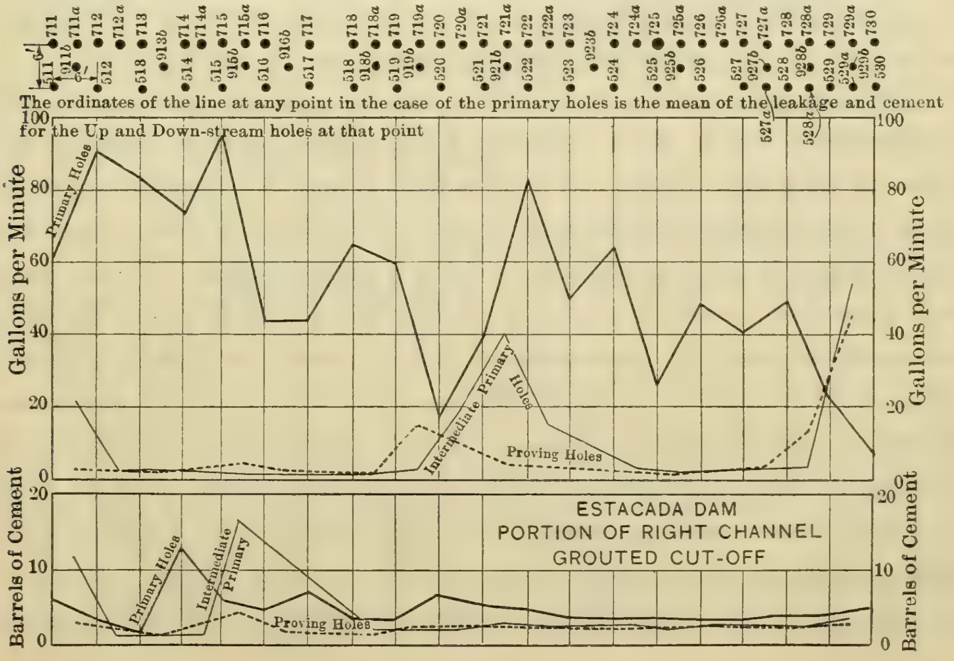


FIG. 5.

Am. Soc. C. E., and representing the consulting engineers, directed that the holes be put down, not through the heel of the dam, as in the left channel, but so as to form a supplementary cut-off immediately up stream therefrom, the plan being to set the casings in the bare conglomerate of the river bottom just back of the cut-off proper, and later, after the grouting and proving were done, to hand-pick a trench to the depth of the casings, namely, 3 ft. or a little deeper. The breccia encountered was sufficiently soft, where speed was not of first consideration, to be removed in this way. This supplementary trench would then be filled with concrete which, bonding with the heel of the dam on one side and with its bottom resting on the curtain-wall supposed to be formed by the grout, would effectually prevent seepage under the dam.

The first grouting was done through casings set in accordance with this plan. However, as in the left channel, the surface material was so lacking in solidity that this method of procedure was not found to be satisfactory, and the plan was modified to the extent of excavating and filling the supplementary cut-off trench before drilling, which was then done through casings set in the concrete. It is apparent that, for purposes of comparison, it is improper to consider the holes tested through casings set in the river bottom with those set in the concrete, as in that case it is impossible to determine how much of the tightening is due to the concreting and how much to the grouting.

The leakage from the holes in which the casings were set in the conglomerate was so great that Mr. F. R. Fisher, of Philadelphia, the Resident Engineer on the work, decided to put down additional primary holes intermediate between those first drilled. By this time, too, it was found to be poor practice to have a large number of holes down and ungrouted at the same time. From this time on, the drills were separated as much as circumstances would permit, and each hole was tested and grouted as soon as drilled, after which several days were allowed to pass before the drilling of any nearby hole was attempted.

Considering a portion of this right channel cut-off, 114 ft. long, in which there were put down and tested thirty-nine primary holes with casings set in the river bed, eighteen primary holes, and twelve proving holes with casings set in the concrete, the following results are noted:

	Gallons per Minute. Maximum.	Minimum.	Average.
39 First primary holes, casings set in river bed....	100	1.0	53.7
18 Second primary holes, casings set in concrete.....	63	1.0	13.0
12 Proving holes, casings set in the concrete.........	44.5	1.2	7.95

The 39 holes first noted, it should be stated, were tested from a tank at Elevation 430, or 46 ft. above the crest of the spillway, and giving a mean head of 182 ft. on the bottom of the hole; the other tests in this, and in the Island and Left Channel Sections following, were made from a tank at Elevation 394, or 7 ft. above the spillway crest, and giving a mean head of 146 ft. on the bottom of the hole.

How much of the tightening, from an average of 54 gal. per min. for the 39 holes drilled and tested before the concrete was placed, to an average of 12.7 gal. per min. for the 18 holes drilled and tested after the concrete was placed, was due to the concrete can never be known. An idea of the relative effect of the concrete and the grout may be deduced by considering an adjoining though somewhat tighter section in which all drilling and testing followed the placing of the concrete. The report for this 54-ft. section is as follows:

	Gallons per Minute. Maximum.	Minimum.	Average.
18 Primary holes, casings set in the concrete..........	17	0.6	4.4
4 Proving holes, casings set in the concrete..........	2.8	0.8	2.2

A comparison of the two tables indicates that but a small percentage of the tightening from 54 to 12.7 gal. per min. in the case of the 114-ft. section was due to the grouting.

Seepage Test.

During construction in the right channel the water above Coffer-dam No. 2 stood at a mean elevation of 317 ft.; behind the coffer-dam, by using pumps, it was held at Elevation 295.

Near the middle of this channel a depression some 30 ft. wide was found in the conglomerate rock. On the cut-off line this was 15 ft.

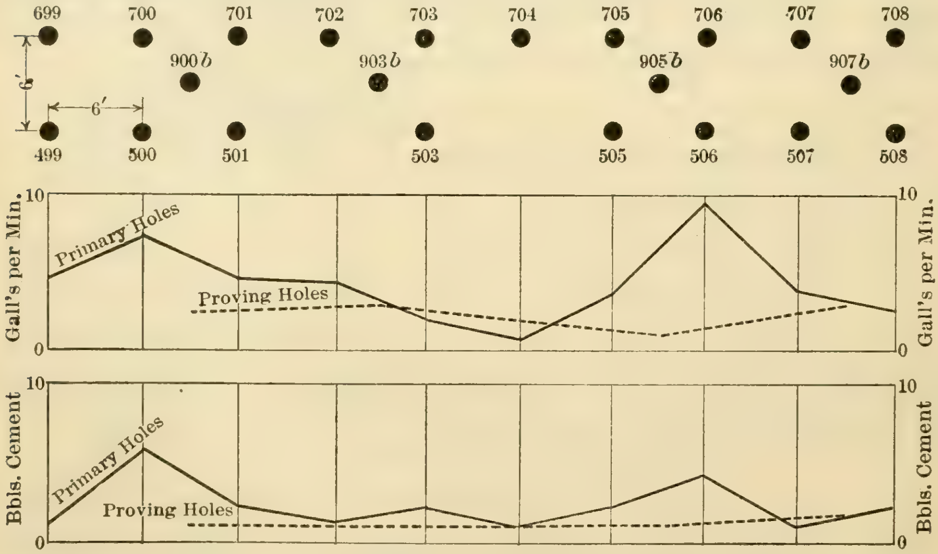
PRIMARY HOLES

Hole No.	699	499	700	500	701	501	702	703	503	
Test Date	6/4'11	6/26'11	6/2'11	5/26'11	5/3'11	6/13'11	5/28'11	5/31'11	5/25'11	
Gall's per Min.	4.4	4.7	2.5	11.9	5.8	3.2	4.3	1.2	2.5	
Grout Date	6/4'11	6/26'11	6/2'11	5/27'11	5/31'11	6/13'11	5/29'11	5/31'11	5/27'11	
Bbls. Cement	0.75	1.25	1	10.5	2.75	1.75	1.25	0.75	4	
Hole No.	704	705	505	706	506	707	507	708	508	Average
Test Date	5/27'11	5/30'11	5/28'11	5/24'11	5/26'11	6/5'11	6/22'11	6/6'11	6/25'11	
Gall's per Min.	0.6	3.2	3.6	17	1.7	3.3	4	1.7	3	4.4
Grout Date	5/27'11	5/30'11	5/29'11	5/25'11	5/26'11	6/5'11	6/22'11	6/7'11	6/25'11	
Bbls. Cement	1	0.75	3.75	7.5	1	1	1.25	1	3.5	2.486

PROVING HOLES

Hole No.	900 *b*	903 *b*	905 *b*	907 *b*	Average
Test Date	6/20'11	6/20'11	6/12'11	6/29'11	
Gall's per Min.	2.5	2.8	.8	2.7	2.2
Grout Date	8/24'11	8/13'11	8/13'11	8/13'11	
Bbls. Cement	1	1	1	1.75	1.187

The ordinates of the line at any point in the case of the primary holes is the mean of the leakage and Cement for the Up and Down-stream holes at that point



ESTACADA DAM
PORTION OF RIGHT CHANNEL
GROUTED CUT-OFF

FIG. 6.

deep, measured from the average level of the surrounding rock, but on the center line of the dam, 80 ft. down stream, this depth had increased to 35 ft. At some earlier age this depression had been worn smooth by the current, but at the time of building the dam it was

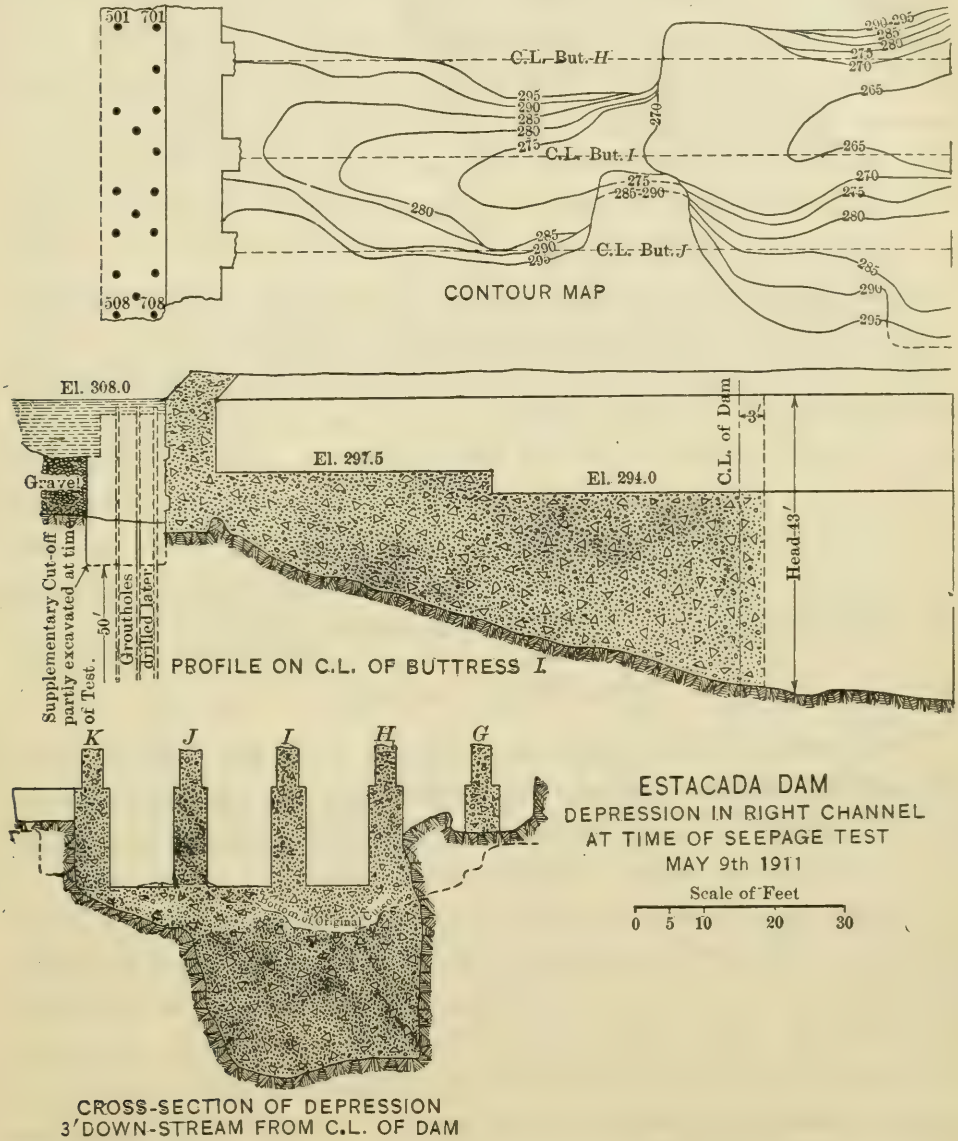


FIG. 7.

filled with loose sand, gravel, and hard boulders. This accumulation was removed, and, as a foundation for the buttresses at this point, mass concrete to a depth of several feet was placed over the entire bottom.

On May 9th, 1911, at which time the concrete had been placed in this gorge to a point 3 ft. below the center line of the dam, and when no drilling and grouting had been done along the cut-off line across the head of the same, the pumps were stopped and the water was allowed to come to Elevation 308, at which point it stood for 6¾ hours. This increased the hydrostatic head between the heel of the dam and the bottom of the depression from 30 to 43 ft. Prior to this "seepage test", no leakage was observed through seams in the bed-rock, and during the test, there was very little, if any. It was raining at the time and the rain water running into the hole precluded making any quantitative measurements. Mr. Fisher, who made very careful personal observations during the test, stated in his report that 2 gal. per min. would be a liberal estimate of the total seepage.

Notwithstanding the small seepage shown by this test, the material was judged to be so erratic that it was decided to continue the drilling and grouting across the head of the depression. The report in detail of the holes put down across the head of this depression will be found on Fig. 6 and summarized in the tabular statement covering the 54-ft. section. From these it will be seen that these holes were, as the seepage test would indicate they should be, comparatively tight.

The Island Section.

That portion of the Island Section next to the left channel showed considerable leakage, though the portion next to the right channel was comparatively tight. Here the conglomerate was fairly firm, and it was possible to grout through casings set therein. So small was the leakage that it was not deemed necessary to hand-pick the supplementary cut-off, but the casings, after grouting, were filled with thick grout and left as shown by the cross-section. It will be noted that in cutting the island to the slope of the deck nearly 50 ft. of the rock in height had been taken away, which accounts for the material being somewhat more solid than at other points. Twenty-three outside and six proving holes put down in this portion show leakage as follows:

	Gallons per Minute.		
	Maximum.	Minimum.	Average.
14 First primary..........	8.5	1.4	4.1
9 Second primary........	6.6	0.7	2.4
6 Proving	2.3	0.9	1.2

FIRST PRIMARY

Hole No.	666	466	667	467	668	468	669	
Test Date	5/10'11	5/13'11	5/15'11	5/8'11	5/21'11	5/17'11	5/9'11	
Gall's per Min.	2	1.4	4.2	4	4	1.6	2.5	
Grout Date	5/11'11	5/14'11	5/15'11	5/9'11	5/21'11	5/17'11	5/12'11	
Bbls. Cement	1	0.5	1.5	50.25	1	1.25	1.25	
Hole No.	469	670	470	671	471	672	472	Average
Test Date	5/13'11	5/16'11	5/15'11	5/9'11	5/9'11	5/13'11	5/13'11	
Gall's per Min.	3.5	6.5	3.5	8.5	5	4	7	4.1
Grout Date	5/14'11	5/16'11	5/15'11	5/14'11	5/13'11	5/14'11	5/13'11	
Bbls. Cement	1.25	10	2.5	3.5	1	2	19	6.85

SECOND PRIMARY

Hole No.	666*a*	466*a*	667*a*	467*a*	468*a*	670*a*	470*a*	671*a*	471*a*	Average
Test Date	5/9'11	5/20'11	5/24'11	5/22'11	5/19'11	5/20'11	5/18'11	5/16'11	5/18'11	
Gall's per Min.	0.7	0.9	1.5	1.5	5.5	1.5	1.6	6.6	1.7	2.4
Grout Date	5/19'11	5/21'11	5/24'11	5/22'11	5/19'11	5/20'11	5/18'11	5/16'11	5/18'11	
Bbls. Cement	1	1.25	1.5	1.5	1.25	1	1.5	1.25	1.25	1.277

PROVING HOLES

Hole No.	866*b*	867*b*	868*b*	869*b*	870*b*	871*b*	Average
Test Date	6/5'11	6/5'11	6/8'11	6/8'11	6/12'11	6/12'11	
Gall's per Min.	0.9	0.9	1	1	1.3	2.3	1.2
Grout Date	8/31'11	9/5'11	8/31'11	8/31'11	8/30'11	8/28'11	
Bbls. Cement	1	1	1	1	1	1.25	1.04

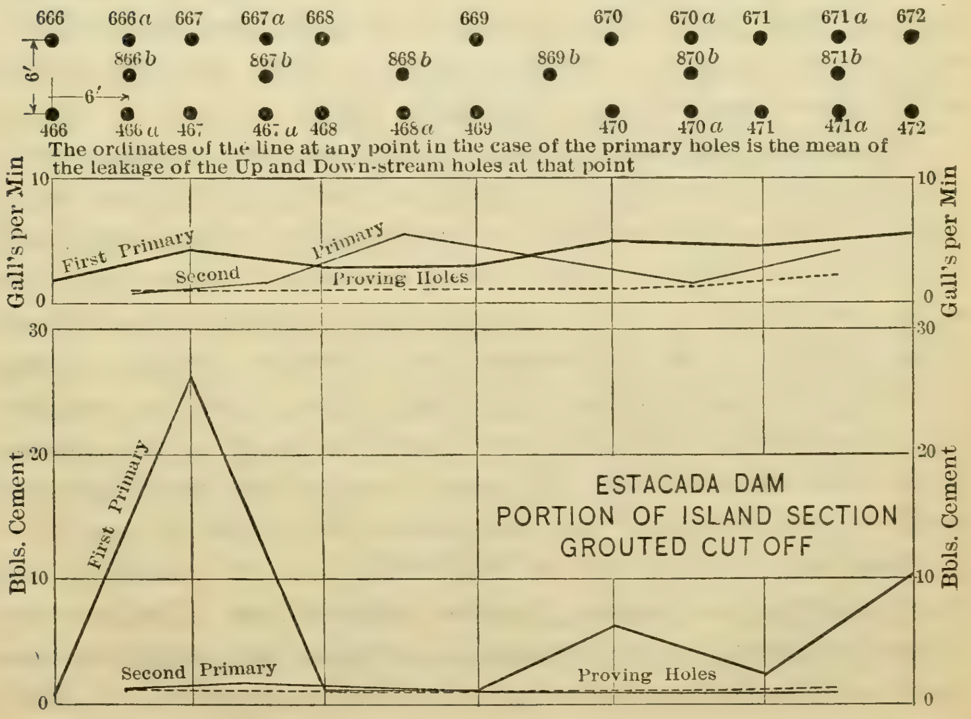


FIG. 8.

The same holes took the following quantities of cement:

	Barrels of Cement.		
	Maximum.	Minimum.	Average.
14 First primary...........	50.25	1	6.85
9 Second primary.........	1.50	1	1.27
6 Proving	1.25	1	1.05

Return to the Left Channel.

In the spring of 1911 several proving holes put down in the left channel and tested with the pump, as had been the original holes, notwithstanding the great quantity of cement that had been forced in, showed leakage and intercommunication between holes practically the same as for the holes put down before any grouting was done. When this was discovered, Mr. Fisher decided to put down new primary holes between those drilled the previous fall. These were tested every 10 ft. as drilled. These tests were so remarkable that the result of the work at this point will be discussed in detail.

By referring to the profile it will be seen that at depths varying from 12 to 40 ft. below the concrete, the normal conglomerate gave place to a water-bearing stratum. Through this the drills made good progress, but obtained a very small percentage of core. The washings from the drills, which were saved in great quantities, appeared as a fine sand of the color of brick or garnet. There was little caving, and the holes remained open after being drilled.

Disregarding the holes put down in 1910 and tested with a pump, and confining our attention to the holes put down in 1911, it will be noticed from the record sheet that the primary holes on the down-stream line, with one exception, were grouted before any testing was done on the up-stream line. The up-stream line, then, had the advantage of the grouting of the down-stream line. Consider, further, a group as made up of four of these new primary holes. It will be seen that the proving holes in each group would have the benefit of the grouting of the four primary holes; the second proving hole would have the benefit of the grouting of the four primary holes plus the one proving hole; and the last proving hole would have the benefit of the grouting of the four primary, plus two proving holes. The fact that the first primary holes were staggered, and those put down later were placed opposite, lends some confusion to the grouping, but, in effect,

still disregarding the holes put down in 1910, the drills were passed over the ground five times. This being the case, the holes drilled in each successive passage should show decreased leakage. That this progressive tightening did not follow and that the results were somewhat complicated and disappointing is shown graphically by the curves of Figs. 9 and 10. Tabulated, the averages appear as follows:

	Average Leakage of Holes, in Gallons per Minute, at Depths of:				
	10 ft.	20 ft.	30 ft.	40 ft.	50 ft.
Down-stream primary	10.3	15.0	29.9	33.2	47.3
Up-stream "	3.05	4.3	14.7	47.2	56.9
First proving	1.4	3.9	15.5	50.7	63.6
Second "	1.6	5.1	13.7	22.1	31.7
Final "	2.0	3.8	6.5	15.0	35.6

Cement was forced into these holes as follows:

	Totals.	Averages.
14 Down-stream primary	117.75 bbl.	8.38 bbl.
13 Up-stream "	79.25 "	6.09 "
11 First proving	21.50 "	1.95 "
12 Second "	20.25 "	1.84 "
17 Final "	26.50 "	1.55 "
67 Holes in all	264.75 "	3.95 "

If to the above are added the 28 holes put down in 1910, taking 172.5 bbl. of cement, and the 7 proving holes put down in the spring of 1911, tested with pump, and not in the above, taking 17.75 bbl. of cement, we have for this 84-ft. stretch of cut-off a total of 102 holes, aggregating 5 050 ft. of drilling, and into which was forced 455 bbl. of cement.

The average leakage of the final proving holes at each different depth, as shown by the foregoing table, expressed as percentages of the average leakage of the down-stream primary holes at the same depth appears as follows: At 10 ft., 20%; at 20 ft., 25.3%; at 30 ft., 21.7%; at 40 ft., 45.1%; at 50 ft., 75.2 per cent.

Back-Flow.

More strange, possibly, than the persistence with which the leakage continued, in spite of all the cement forced in, was the fact that with nearly all these holes, as with those put down in 1910, a very

DOWN-STREAM PRIMARY

Hole No.	637 *a*	638 *a*	639 *a*	640 *a*	641 *a*	642 *a*	643 *a*	644 *a*	645 *a*	646 *a*	647 *a*	648 *a*	649 *a*	650 *a*	Average
Test Date	9/8'11	9/29'11	9/15'11	8/26'11	8/22'11	8/29'11	9/1'11	8/19'11	8/30'11	8/24'11	8/30'11	8/23'11	8/29'11	8/20'11	
Gall's per Min	8	10	5	5	127	36.9	58.8	145	89.5	45.5	30	30	35.5	35.5	47.3
Grout Date	9/10'11	9/29'11	9/20'11	8/26'11	8/23'11	8/29'11	9/2'11	8/19'11	8/30'11	8/24'11	8/30'11	8/23'11	8/29'11	8/21'11	
Bbls. Cement	2	1.5	1.25	27	18.75	28	5	6.5	6.25	5	8.75	1.25	1	5	8.375

UP-STREAM PRIMARY

Hole No.	438 *a*	439 *a*	440 *a*	441 *a*	442 *a*	443 *a*	444 *a*	445 *a*	446 *a*	448 *a*	449 *a*	450 *a*	450 *b*	Average
Test Date	10/2'11	9/29'11	9/2'11	9/7'11	9/4'11	8/25'11	9/6'11	9/3'11	9/7'11	9/3'11	9/5'11	9/1'11	9/23'11	
Gall's per Min.	36.8	29	11.6	31	40.2	35.5	44.5	75	137	102	135	16.8	45.5	51.0
Grout Date	10/2'11	9/29'11	9/2'11	9/7'11	9/4'11	8/25'11	9/6'11	9/4'11	9/9'11	9/3'11	9/6'11	9/1'11	9/23'11	
Bbls. Cement	1.75	1.75	22	2.75	3	3	2.5	6.25	3.75	10	4.25	16.25	2	6.09

FIRST PROVING

Hole No.	839 *b*	840 *a*	841 *a*	842 *a*	843 *a*	843 *b*	844 *b*	845 *b*	847 *c*	848 *c*	850 *a'*	Average
Test Date	10/7'11	10/6'11	9/14'11	9/11'11	9/22'11	9/27'11	9/15'11	8/12'11	9/6'11	9/12'11	9/11'11	
Gall's per Min	4.7	41	26	45.5	135	10.9	89	135	59	68.7	121.5	67.0
Grout Date	10/7'11	10/6'11	9/20'11	9/11'11	9/23'11	9/27'11	9/20'11	9/13'11	9/19'11	9/13'11	9/11'11	
Bbls. Cement	1.5	1.75	2	2	2.5	1.75	2	1	2.5	2.5	2	1.95

SECOND PROVING

Hole No.	839 *a*	840 *b*	841 *b*	842 *b*	843 *c*	844 *c*	845 *c*	846 *c*	847 *b*	848 *a*	849 *a*	Average
Test Date	10/5'11	10/3'11	9/25'11	9/30'11	10/6'11	10/3'11	9/24'11	9/22'11	9/27'11	10/4'11	10/4'11	
Gall's per Min	27	39.1	25.4	20.1	33.5	29	15.6	78.5	18	30.5	32	31.7
Grout Date	10/5'11	10/3'11	9/25'11	9/30'11	10/6'11	10/3'11	9/24'11	9/22'11	9/27'11	10/4'11	10/4'11	
Bbls. Cement	1.5	1.75	1.75	2	1.75	1.75	1.5	1.5	1.75	2	3	1.84

FINAL PROVING

Hole No.	837 *b*	838 *c*	839 *c*	840 *c*	841 *c*	842 *a'*	842 *c*	843 *a'*	844 *a*
Test Date	10/11'11	10/13'11	10/12'11	10/12'11	10/11'11	10/13'11	10/11'11	10/13'11	10/10'11
Gall's per Min	5.5	8.5	22.3	37.5	33	45.5	46.5	30.7	60.5
Grout Date	10/13'11	10/15'11	10/15'11	10/15'11	10/15'11	10/15'11	10/15'11	10/16'11	10/15'11
Bbls. Cement	2	1.25	1.25	1.25	3	2.75	1.5	1	1.75
Hole No.	845 *a*	845 *b'*	846 *a'*	847 *a*	848 *b*	848 *b'*	849 *b*	850 *c*	Average
Test Date	10/10'11	10/14'11	10/10'11	10/10'11	10/11'11	10/15'11	10/11'11	10/11'11	
Gall's per Min	35.5	34	32.9	33	9.5	73	76	21.8	35.6
Grout Date	10/15'11	10/16'11	10/16'11	10/16'11	10/15'11	10/16'11	10/16'11	10/16'11	
Bbls. Cement	1	1.25	1.75	1.25	1.5	1.5	1.5	1	1.55

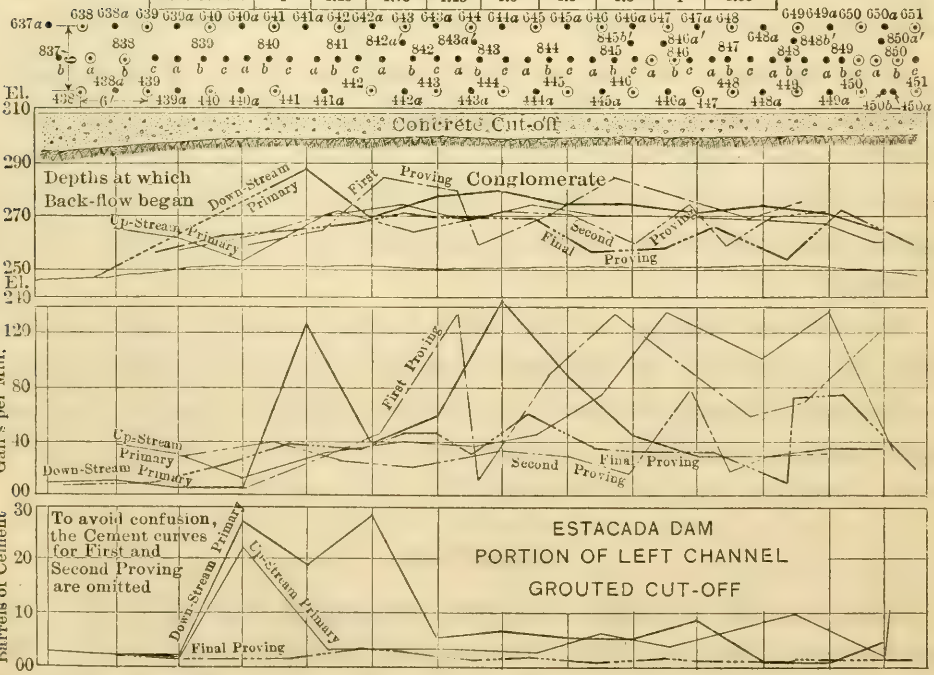


FIG. 9,

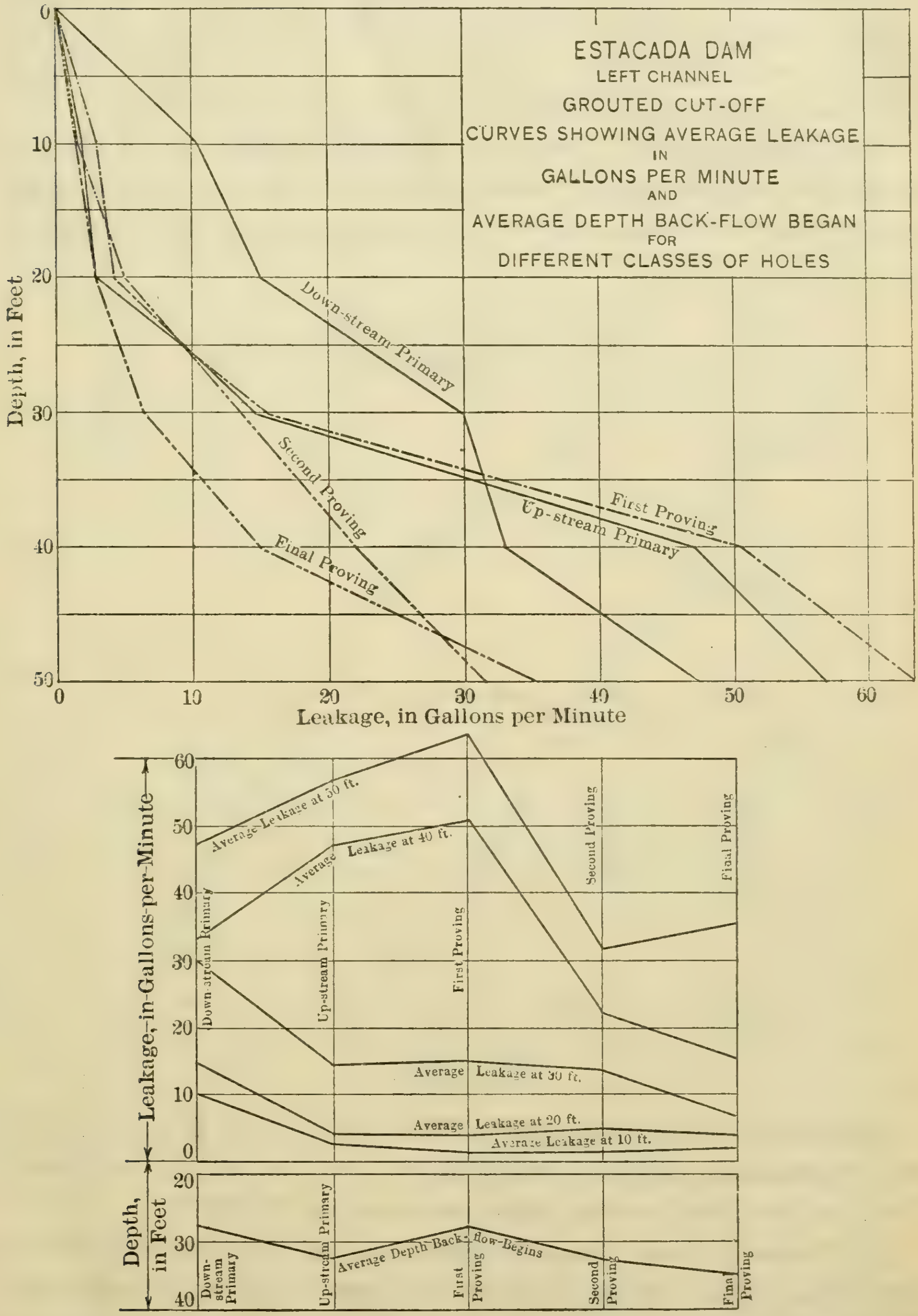

ESTACADA DAM
LEFT CHANNEL
GROUTED CUT-OFF
CURVES SHOWING AVERAGE LEAKAGE
IN
GALLONS PER MINUTE
AND
AVERAGE DEPTH BACK-FLOW BEGAN
FOR
DIFFERENT CLASSES OF HOLES
Down-stream Primary
Up-stream Primary
First Proving
Second Proving
Final Proving
Depth, in Feet
Leakage, in Gallons per Minute
Average Leakage at 50 ft.
Average Leakage at 40 ft.
Average Leakage at 30 ft.
Average Leakage at 20 ft.
Average Leakage at 10 ft.
Leakage, in Gallons per Minute
Depth, in Feet
Average Depth Back-flow Begins
Down-stream Primary
Up-stream Primary
First Proving
Second Proving
Final Proving

FIG. 10.

insignificant head produced a back-flow from the river. At this time the construction of the dam had completely blocked the right channel, so that operations were carried on in the left, behind coffer-dams which blocked off first one and then the other half of this channel, and from these resulted the head which produced the flow.

The average depth at which the back-flow commenced in the case of the down-stream primary holes was 27.2 ft.; the up-stream primary, 32.0 ft.; the first proving, 27.7 ft.; the second proving, 32.5 ft., and the final proving, 35.0 ft.

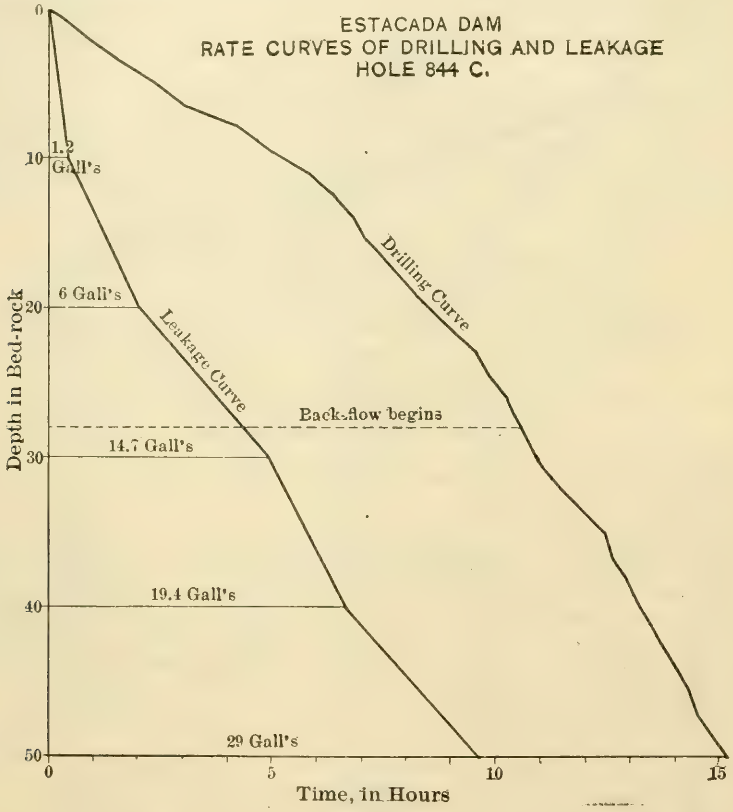


FIG. 11.

The depth below the bottom of the concrete cut-off at which this flow commenced, the head producing it, and the gallons per minute for 14 of the final proving holes, are given in Table 1.

In Table 1, under head, is given the vertical distance that the top of the casing is below the level of the water against the coffer-dam. This small head, averaging only 4.4 ft. for these 14 holes, produced a back-flow averaging 12.7 gal. per min., or one-third of the average leakage for the same holes when tested from a tank some 70 ft. above

the level of the water against the coffer-dam. In one hole, 845*a*, the back-flow, under a head of 7.7 ft., was 30 gal. per min., though the same hole, under test, leaked only 35.5 gal. per min.

TABLE 1.

Hole No.	Depth at which back-flow commenced.	Head.	Gallons per minute at full depth.
837*b*	..	....	0.0
838*c*	40	2.0	4.0
839*c*	35	5.6	13.0
840*c*	35	5.8	15.0
841*c*	32	2.9	11.0
842*c*	30	4.0	14.5
844*a*	30	6.7	17.0
845*a*	42	7.7	30.0
847*a*	33	7.2	22.0
848*b*	46	6.6	1.8
849*b*	28	1.0	8.0
850*c*	40	1.0	19.0
843*a'*	30	4.0	18.0
846*a'*	42	6.7	4.0
Average........	33	4.4	12.7

Not only did the leakage continue and the back-flow persist, but the communication between holes was present even to the 17 final proving holes. In fact, by a system of piping, 13 of these final holes were connected and tested at one time. This, as well as other combination or multiple tests at other points of the cut-off, showed much less average leakage per hole than the average of the same holes tested singly. However, the fact that 17 holes put down along a stretch of cut-off 84 ft. long, in which 84 holes before them had taken 427.5 bbl. of cement, could communicate so freely that pressure from a tank on the crest of the dam applied to any of them would cause water to flow from the other 16 was poorly offset by the fact that 13 of these same holes when tested in combination showed an average leakage of 17.5 gal. per min. as against an average of 34.4 gal. when tested individually. This reduced leakage in the cases of the combination tests was interpreted as indicating that several holes intercepted the same seam or seams.

Grout in the Cores.

At one point, where a trench was excavated subsequent to the grouting, the cement could be traced as a white thread completely filling a seam ½ in. wide for 38 ft. This, of course, was only a few

feet beneath the surface. At greater depths the only evidence of the diffusion, aside from the decreased leakage in testing, would be the cement brought up as core or in the cement flakes in the wash from the drills. The grayish color of the wash, indicating that the drill was penetrating grout, was of much more frequent occurrence than the finding of solid grout in the cores. Of course, where the drilling of the proving holes followed in a few days or weeks the drilling of the primary holes, it was to be anticipated that the cement would be ground up by the attrition of the bits, the shot, and the harder particles of rock. After a lapse of several months, cement core might be obtained, but even then it would be best to use a double-tube core barrel. However, all cores were carefully inspected for any traces of cement, and, though some *bona fide* samples were found, they were very rare.

The words "*bona fide*" are used advisedly. At one time, owing to the poor showing being made by the grouting, there was serious thought of discontinuing it altogether. At this juncture, as the writer discovered later, at least one driller, moved by a commendable desire to see his job continue, joined short pieces of core with cement, which sample, deposited in the core box, was later seized on and carried up to the office as conclusive evidence of the diffusion of the grout.

Tests After Completion of Dam.

For a period of nearly 2 weeks after the pond had been allowed to fill, to admit of completing the excavation of the tail-race, the sluice-gates would be opened at night and closed during the day until such time as the water was nearing the crest. This, save for pools and pot-holes, left the left channel dry for several hours each day, and during these intervals careful daily inspections were made to detect any springs resulting from the 80 to 90-ft. depth of water above the dam. None was found, and the only seepage of any kind was very small in quantity, coming from well up on the left slope. The dam was completed at the time the fall rains set in, and this very insignificant seepage was believed to be due to the filling of the swales and depressions which occurred at intervals in the nearly level bench topping the left bank at the site of the dam.

When the water was flowing over the crest, a core drill was taken inside the dam and two holes put down. The first of these was next

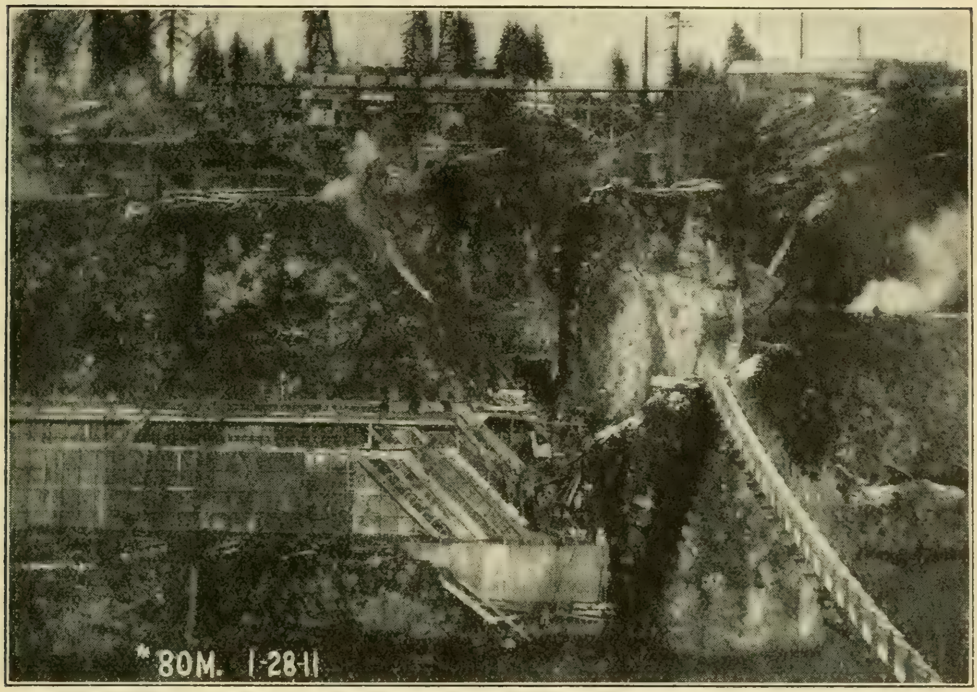


FIG. 12.—VIEW FROM LEFT BANK. ISLAND BEING CUT TO SLOPE OF DAM.

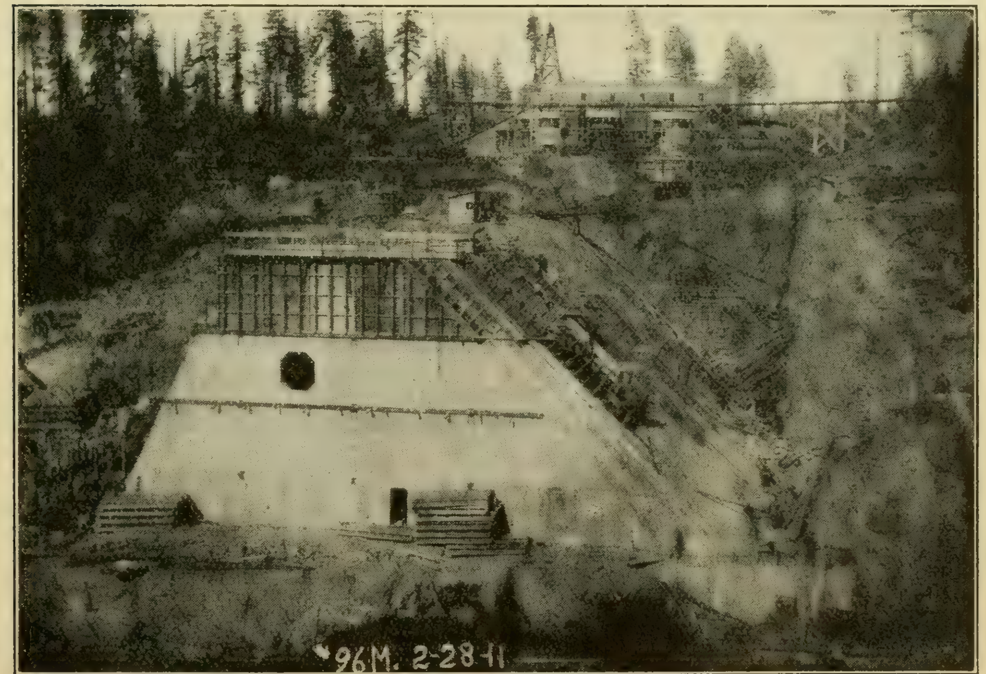


FIG. 13.—VIEW FROM LEFT BANK. DECK BEING EXTENDED TO INCORPORATE ISLAND IN DAM.

the left channel on what had originally been the island, and at a point 35 ft. down stream from the lower side of the grouted cut-off. Table 2, which is an abridgment of one appearing in Mr. Fisher's report to the owners of the plant, shows that water commenced to flow when the drill had penetrated the rock to a depth of 28.5 ft.

TABLE 2.—Hole No. 1 000.

Elevation, top of casing........................328.2
" of bottom of rock at hole..............325.0
" bottom of concrete cut-off opposite hole..306
" grouted cut-off opposite hole............248

Elevation, bottom of hole.	Elevation, water in pond.	Flow over top of casing, in gallons per minute.
296.5	375	First trace of flow.
289.5	376	0.9
280.5	387	2.2
270.5	387	5.4
260.5	387	5.4
249.0	387	6.6
245.5	387	7.5

The head, or difference of level, between the top of the casing and the water in the pond, with the exception of the first two determinations, is constant, and amounts to 59 ft.

A few days later, when the water in the pond stood at Elevation 388.4, the casing was extended upward with 1¼-in. pipe, great care being taken to have all joints tight. When the water finally came to rest it was found to be at Elevation 364.2 or 24.2 ft. lower than the water passing over the crest above.

The other hole was put down directly in the left channel, and opposite that point in which the leakage had been most serious and the rock most refractive to tightening by the grouting process. This hole, No. 1 001, was opposite Hole No. 644*a* and 30 ft. down stream from the lower line of the cut-off. The first trace of flow from this hole was at a depth of 22.9 ft. in the rock.

In this hole the casing was extended and the height to which the water would rise was determined at intervals as the drilling progressed.

In this case the difference between the top of the casing, the point at which the flow was measured, and the elevation of the pond was nearly constant, and amounted to 79.5 ft.

TABLE 3.—Hole No. 1 001.

Elevation top of casing..........................315.1
" of rock at hole.........................308
" bottom of concrete cut-off opposite.......302
" " " grouted " " 248

Elevation, bottom of hole.	Elevation, water in pond.	Flow, in gallons per minute.	Elevation at which water came to rest.
285.1	....	First trace of flow.	315.1
283.1	387.5	0.7	368.0
281.6	387.1	16.9	369.4
270.1	387.8	26.0	369.6
255.1	387.5	32.9	369.7
250.1	387.1	46.0	373.1
247.1	387.1	53.8	374.0

In this hole, when the bottom was at Elevation 282.6, or 25.4 ft. in bed-rock, the drill suddenly dropped 2 in. and the water came up in greatly increased force and volume.

TABLE 4.—Cost Data.

Quantities:		
Total drilled, 555 holes....................................		34 038 lin. ft.
Average per drill per 10-hour shift..........................		13.2 " "
Average shot per drill per shift.............................		1.1 lb.
Primary or outside holes, grouted............	375 taking	1 526.50 bbl. cement.
Proving or middle holes, grouted..............	160 "	275.25 "
Tight holes filled................................	12 "	15.00 "
Holes lost..	8 "	...... "
Setting casings, etc............................	.... "	125.25 "
Total..	555 taking	1 942.00 bbl. cement.
Most cement in any one hole..........................50 bbl.		
Average for 535 holes taking grout.................... 3.37. bbl.		
Cost:		
Labor, drilling......................................	$19 842.60	$0.59 per lin. ft.
Labor, grouting	6 285.32	0.18 "
Cement, at $2.20 per bbl. at cut-off............	4 272.40	0.12 "
Repairs, oil, waste, shot, etc..................	5 908.35	0.17 "
Depreciation on grouting plant, 50%..........	5 116.70	0.15 "
Total direct..................................	$41 425.37	$1.21 per lin. ft.
Other charges were prorated as follows:		
General plant, camp, etc........................	$15 304.63	$0.45 per lin. ft.
Coffer-dams and pumping........................	5 121.63	0 15 "
Engineering and superintendence.............	6 414.91	0.19 "
Total cost....................................	$68 266.54	$2.00 per lin. ft.

Conclusions.

In a piece of work of this character and magnitude, where much is experimental, nothing would be gained in attempting to present a

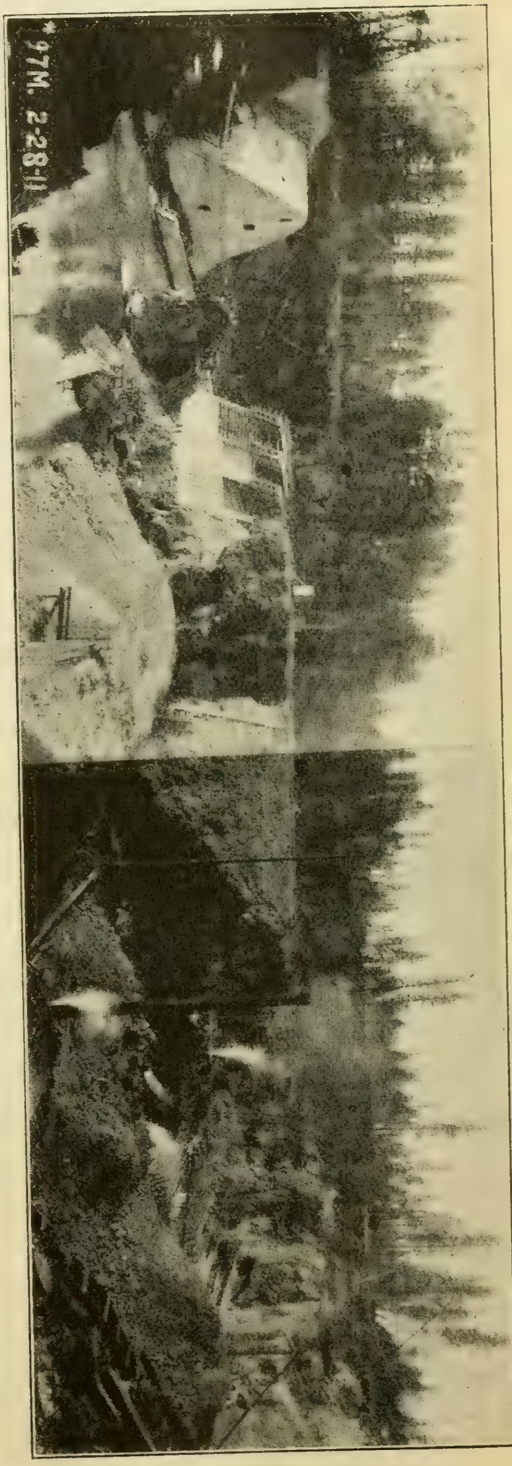


FIG. 14.—GENERAL VIEW, LOOKING DOWN STREAM. ISLAND CUT TO SLOPE OF DAM.

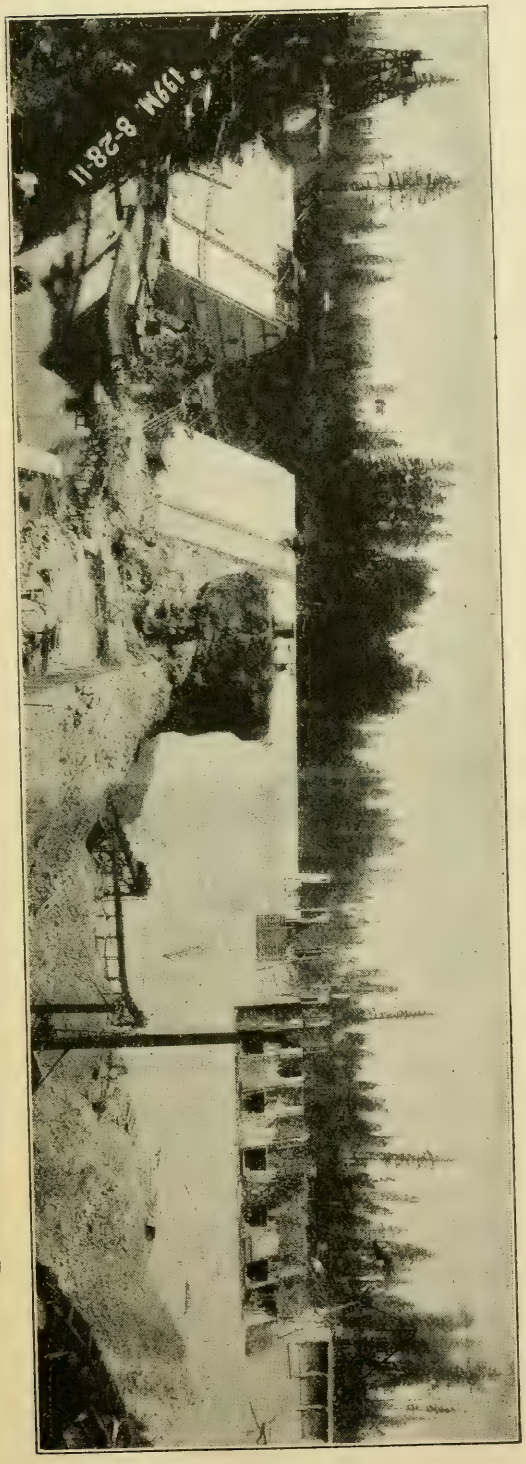


FIG. 15.—GENERAL VIEW, LOOKING DOWN STREAM, AFTER ISLAND HAD BEEN INCORPORATED IN DAM.

review or an epitome of the work as a whole, such, for example, as giving the average leakage of all the primary holes and contrasting this with the average for all the proving holes. Such a marshaling would impart, not information, but misinformation. One of the reasons for this is that any hole put down near one already grouted is in a sense a proving hole, whether in the middle or in an outside line of holes. Other reasons are: that the testing was done in part by pump and in part by gravity; in part from a tank at one elevation and in part from a tank at a different elevation; in part through casings set in the conglomerate before the cut-off trench was concreted and in part through casings set in the concrete with which the trench was filled. For these reasons, it seems best to rest the case with the consideration of the several portions of the work taken up in the preceding pages. These cover portions of the cut-off where the material was best as well as where it was poorest; where the grouting was most effective as well as where its benefits were least marked.

Several engineers from different parts of the country, who visited the work during the closing days of construction, attached considerable importance to the great flow from the two holes put down inside the dam, and to the fact that the pressure encountered was sufficient to raise the water to within a few feet of the crest of the dam. The opinion seemed to be that the pressure and flow proved the failure of the grouting process. To the writer, they mean merely that the water passing through the grouted cut-off at a depth of 25 to 30 ft. is stopped or greatly checked in its flow by the denseness of the rock in front, and is held down by the denseness of the rock above. The pressure and flow, in view of this, seem to indicate, rather, that the grouting was not needed, though this is assuming that the tightness of the surface rock is in no degree due to the grouting.

The writer is of the opinion that most engineers who have followed this description will have concluded, in so far as this job is concerned, that over a portion of the cut-off no grouting was needed, and that over those parts where it was needed it did little good. The initial tightness over a part of the right channel and island sections, as shown by the seepage test and pressure tests, and the great leakage, communication, and back-flow of the final holes in the left channel afford ample basis for such an opinion. However, calculating for a head of 80 ft. and assuming an efficiency of 50% for power delivered to

the customer, it is found that, at $33 per kw-year, a saving of 26 sec-ft. would pay 7% on the $41 000 representing the direct cost of the grouting. Whether or not this quantity of water has been saved, no one can say. Even had the first primary holes shown very free leakage and the final proving holes tested absolutely tight, it would not have been known, so different are the conditions when water is forced through a casing into a hole 50 ft. in the rock from those when the pond is filled and there is no such penetration to the interior rock.

In rock having continuous seams, the grouting process would probably be much more effective than in the "volcanic débris" of the Clackamas Canyon. In this, though the grout will fill the larger cavities and interstices, it cannot be called a success, in so far as its being an agent for providing means for actually changing the structural character of the formation, as was hoped when beginning the work.

The knowledge gained by the experience at Estacada may be summed up as follows:

(1) Do all drilling, testing, and grouting through casings set in the concrete cut-off.

(2) Do all testing from elevated tanks, and not by pump.

(3) Test and grout each hole as soon as drilled, and for a few days thereafter keep the drills away from the probable zone of diffusion.

(4) In grouting, especially at high pressures, it is best to close the valve before the tank is entirely empty, as the air following the grout into the hole is apt to make trouble.

(5) Begin with a comparatively thin grouting mixture and, if taken freely, thicken until each succeeding batch requires either an increased time for discharging or an increased pressure. To force charge after charge of thin grout into a hole probably means in a great measure the wasting of cement.

The writer's opinion, now, is that either a single row of holes with close spacing or two rows of holes very close together in an up-and-down-stream direction, with casings staggered, is preferable to the triple line used at Estacada. At the Lahontan Dam, of the Truckee-Carson Project,* two rows of casings were put down, the distance be-

* *Engineering Record*, March 29th, 1913, p. 340; and *Engineering News*, April 3d, 1913, p. 647.

tween the rows was only 2 ft., and the distance between the holes of each row, 3 ft. As the casings in the two rows were staggered, it virtually amounted to a casing every 18 in. Over one section of this work, every fourth hole in the up-stream line of casings was first drilled, tested, and grouted. When the end of the cut-off was reached, the drills were returned to the beginning and the middle hole in each space was drilled, tested, and grouted, after which the drills were returned a third and a fourth time, drilling the remaining holes. This made every hole after the first, a proving hole, and as practical tightness was secured with the drilling of the up-stream line, the casings of the down-stream line were not needed, and were drilled through in only four or five instances as final proving holes.

The experience at Estacada indicates that the grouting should in no case be relied on in lieu of the usual concrete cut-off. There are two reasons for this:

(1) The efficiency of grout as a curtain-wall cannot be foretold.

(2) The proper diffusion of the grout can be secured only when the concrete of the cut-off closes the surface seams and confines the pressure to a depth at which it may be effective in tightening the underlying material.

Where very porous material is encountered below the practicable limit of depth for a concrete cut-off which, as in the case of the left channel at Estacada, proves refractive to the tightening by the grouting of isolated holes, the desired end might be secured by drilling a great number of holes close together and by springing the rock with small charges of powder from the bottom up to the top of the water-bearing stratum, thus shattering and loosening up the intervening material so that the grout would then form an impervious barrier.

As already stated, the pressure testing, though affording one means of measuring the effect of the grout, by no means proves it to be necessary.

In the case of a storage dam, the best plan would seem to be to set the casings in an offset cut-off just up-stream from the heel of the dam, as was done along part of the dam at Estacada. Through these casings a hole could be put down every 12 to 16 ft. to make sure that there were no large crevices. These casings should be tested and grouted and the need for further drilling and grouting left to be determined

by the filling of the pond. If the leakage were excessive and the seams did not tighten by the natural silting processes, the pond, at time of minimum storage, could be entirely emptied and the grouting continued. In this case it would be necessary to grout only at those points where seepage showed it to be needed, and, if tightness were secured, the engineer would have absolute proof of the efficiency of the process. Of course, with an earth dam like that at Lahontan, already mentioned, this method is impossible, and grouting must precede the construction of the dam proper.

In the case of a power dam, the same plan might be followed, and drilling and grouting done by drawing off the water at the time of minimum flow. Whether to do this or to carry the grouting along with the building of the dam, as was done at Estacada, is a problem the solution of which depends on what the minimum flow power is worth and whether the load might, without any great inconvenience, be transferred to other plants for a short time.

The consulting engineers on this work were Messrs. Sellers and Rippey, of Philadelphia. They were represented at various times and in various capacities on this and other projects of the Company, including the proposed 150-ft. dam on the Clackamas River, by Messrs. William H. Cushman and H. V. Schreiber, Members, Am. Soc. C. E., and Messrs. W. L. Fitzgerald and S. C. Hulse, Associate Members, Am. Soc. C. E.

A. Gardner, Assoc. M. Am. Soc. C. E., represented the Ambursen Hydraulic Construction Company, of Boston, owners of the hollow-dam patents under which the dam was built, and L. I. Fletcher, Assoc. Am. Soc. C. E., was Superintendent for the contractors, the Puget Sound Bridge and Dredging Company.

Mr. T. W. Sullivan is Chief Hydraulic Engineer of the Portland Railway, Light and Power Company, owners of the plant, and Mr. F. R. Fisher was the Resident Engineer on the dam. The writer was Assistant Engineer of the same Company, and was connected with the work from the time of making the survey to the delivery of power.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE DIVERSION OF IRRIGATING WATER FROM ARIZONA STREAMS.

By A. L. Harris, Assoc. M. Am. Soc. C. E.

To be Presented February 18th, 1914.

Most of the irrigating water obtained from Arizona streams comes from the higher mountains. Very little rain falls in the lower and more level country, where cultivation is practicable. For ages the typical stream has gathered its load of silt, sand, and gravel in these mountains and forced it, by reason of strong slopes, through the canyons and down to the low country, where grades become less, and it is spread out evenly in the broad, gently sloping, cultivable valleys. Usually, such valleys show evidences of having been flowed by slack-water for periods long enough to deposit deep surface beds of fine silt. The overflow has eventually cut its way out through surrounding hills, and worn down the outlet below the valley level. A channel has then been cut across the valley floor in the alluvium, toward which channel the surface slopes and drains on each side. The irrigated valley is now like a great shallow dish, inclined a little, and with the bed of the stream running across it in the direction of its slope. By intercepting the river water with a dam where it enters the valley from the surrounding hills, it may often be diverted into two canals, one to the right and one to the left of the stream. Throughout a wide

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

circuit around the edges of the dished valley the canals are located, with gentle grades to insure the flow and with the water surface generally a few inches above the natural ground to enable the water to be taken out at intervals. Of course, it is then possible to draw water into ditches from one of these canals to irrigate any piece of land between it and the river, as there is always a slope in that direction. As the river seldom flows through the middle of the valley, the area of irrigable land under each main canal is not the same. Indeed, it often happens that practically all the irrigable land of a valley lies on one side of the river. The great Salt River Valley about Phœnix has about twice as much irrigable land on its north side as is found on the south, and the Agua Fria and Buckeye Districts, on the Agua Fria and Gila Rivers, respectively, each utilize practically only one side of the stream.

Were it not that the arid nature of the country still allows erosion phenomena to go on vigorously, the provision of works for diverting the irrigating water would be much simpler. For some years the writer has frequently been called on to design diversion works in connection with both United States Government and private irrigation projects in this region, and this paper deals with the principal features and conditions relative to this experience.

Conditions to be Met.

The natural conditions in the locality of diversion present a stream having a flow which varies between very wide limits, and subject to very sudden changes. The floods often carry large quantities of water in great rushes, bearing down with them all kinds of drift—on the surface, in suspension, and grinding along the bottom—wood, silt, sand, gravel, and boulders. The works will be subject to much heat and dryness, but no ice (except on the plateaus of northern Arizona). The range of surface temperature is 160° or more. It is difficult to find an all rock foundation, as the river bed generally consists of a deep canyon, with rocky sides, which has been nearly filled with drift.

For the requirements of irrigation, diversion works must be such that the flow of water shall not be interrupted during long periods in the growing season, which, by the way, in southern Arizona, is nearly 12 months in the year. Canals should be kept free from sand and gravel, but the fine silt carried in suspension is valued highly by the

farmer, for keeping up the strength of the soil, very little other fertilization being necessary. If the system includes a storage reservoir somewhere up stream, much of this valuable silt, unfortunately, is deposited in the bottom of the reservoir, where, for the most part, it is not only useless but in time becomes a serious problem. The capacity to divert and distribute promptly unusually large quantities of water in flood times is generally required. Surface drift must be kept out of the canals. The layout of the whole works must also be such that the banks of the canals, where near the river, will be protected from the washing of the waste waters when floods pass down the river.

The Diverting Dam.

For diverting water into the canals, some kind of dam is generally required, although in special cases a subterranean collector conduit is used. It is generally necessary to raise the elevation of the water considerably in order to place the canals above the reach of flood water and to raise the surface level in them as much as possible for serving the greatest area of irrigable land.

The original diverting dam for irrigation was merely an obstruction of brush, stone, and earth projecting into the stream (as a wing) or across it, and deflecting the water into a ditch at about the natural level of the stream bed. Any small rise in the river destroyed it, and it had to be rebuilt. The point where the river water entered the canal was generally chosen at a place naturally suited to withstand the wear of flood, as by cutting through a projecting rock ledge or boulder point. The most serious expense and loss with such a canal head was not that of rebuilding the brush and stone dam, but the loss of crops due to the interruption of irrigation. However, by the hard and persistent work of the pioneers, most of the irrigated districts in Arizona were started and developed from such a beginning.

The next step in improvement was by the use of timber and rock-filled crib dams. Although a dam of this type can doubtless be made to perform the required service, its common history in Arizona has been that, because of insufficient protection of the foundations, or by ill chosen dimensions governing discharge, it has been sooner or later breached by a flood, causing heavy losses. In the absence of accurate flood data, and even when such data are at hand, it is difficult for the designer to appreciate the destructive action to be provided against

in damming these streams, except after first-hand experience. Usually, the river presents a very shrunken appearance, because of dry weather, and the intervals between extreme floods, often extending through a considerable number of years, give time to cover up and age the evidences of their action.

In this climate, timber is an even more temporary material than usual. All its possibilities for warping and checking are brought out by wetting followed by much heat and dryness. The necessary iron fastenings are hastened in rusting by alkalies often contained in the water. As this timber must usually come from Oregon and Washington, by way of Southern California, it is also expensive. At the present and prospective prices of cement, the expense of timber-crib dams in this locality is not justified by their enduring qualities.

Some diversion dams have been built of cemented stone masonry, but of late the most suitable and available material has proved to be concrete. Concrete materials are found at or near almost any diversion site. The excavated materials from the foundation trenches often prove to be quite suitable. Thus it appears that, for most cases in this locality, a concrete dam and intake works suitably designed and constructed is, both economically and structurally, about the ideal structure for diverting irrigating water. The concrete dam is usually carried entirely across the stream, but, on account of expense and poor foundation conditions, it sometimes appears advisable to build, next to the intake gates, a wing-dam of permanent construction, which acts as a spillway for moderate floods; and to maintain a dike of earth with a higher crest to a closure on the opposite river bank. This dike is expected to go out in the heavier floods, and then must be replaced.

The main features of the complete diverting dam, which is practically always of the overflow type, are the foundation, the apron, the rollway, the sand sluices, and the canal intakes.

Where bed-rock cannot be secured for foundations, excavation will generally discover beds of heavy boulders deposited by the stream, tightly packed together and chinked in with gravel and sand. On account of the usual position of the diverting dam at the mouth of a canyon, as mentioned previously, the heavier products of erosion coming from the mountains can be confidently looked for in the bed of the river. The deposits grow progressively coarser as the depth increases. A series of rather heavy walls is laid out and built in

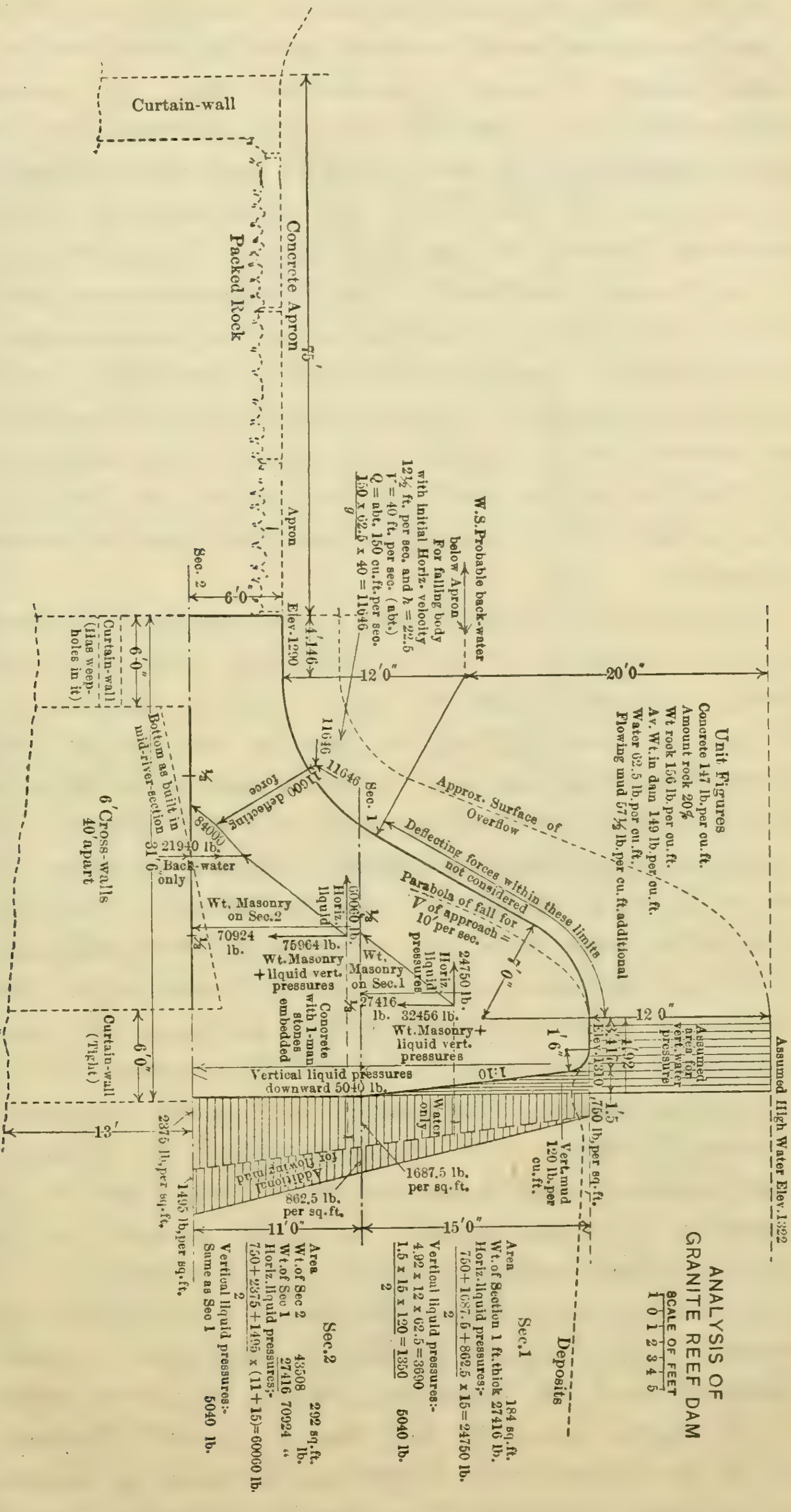

ANALYSIS OF
GRANITE REEF DAM
SCALE OF FEET
1 0 1 2 3 4 5
Curtain-wall
Concrete Apron
Packed Rock
Apron
Elev. 1290
W.S. Probable back-water below Apron
For falling body with initial Horiz. velocity 12½ ft. per sec. and h = 22.5
V = 40 ft. per sec. (abt.)
Q = abt. 150 cu.ft. per sec.
Unit Figures
Concrete 147 lb. per cu. ft.
Amount rock 20%
Wt rock 156 lb. per cu. ft.
Av. Wt. in dam 149 lb. per cu. ft.
Water 62.5 lb. per cu. ft.
Flowing mud 57½ lb. per cu. ft. additional
Approx. Surface of Overflow
Deflecting forces within these limits not considered
Parabola of fall for V of approach = 10' per sec.
11600 deflecting force
Sec. 1
Sec. 2
Curtain-wall (Has weep-holes in it)
Curtain-wall (Tight)
Bottom as built in mid-river-section
6' Cross-walls 40' apart
Back-water only
Wt. Masonry on Sec. 2
Wt. Masonry on Sec. 1
Wt. Masonry + liquid vert. pressures
Concrete with 1-man stones embedded
Vertical liquid pressures downward 5040 lb.
Assumed area for vert. water pressure
Assumed High Water Elev. 1322
Elev. 1310
Water only
Additional for flowing mud
Vert. mud 120 lb. per cu. ft.
Deposits
Sec. 1
Area 184 sq. ft.
Wt. of Section 1 ft. thick 27416 lb.
Horiz. liquid pressures:-
(750+1687.5+862.5)/2 x 15 = 24750 lb.
Vertical liquid pressures:-
4.92 x 12 x 62.5 = 3690
(1.5 x 15 x 120)/2 = 1350
5040 lb.
Sec. 2
Area 292 sq. ft.
Wt. of Sec 2 43508 lb.
Wt. of Sec 1 27416 70924 "
Horiz. liquid pressures:-
(750+2375+1495)/2 x (11+15) = 60060 lb.
Vertical liquid pressures:-
Same as Sec 1 5040 lb.

FIG. 1.

trenches excavated down to the best gravel or boulder bed within reach. The dam is supported mostly on these walls, but is assisted by the undisturbed masses of gravel lying between them. The arrangement of walls consists of a deep one, put in by timbering and pumping, running lengthwise of the dam under the heel, and a somewhat shallower one under the toe. These two main walls are connected by cross-walls at intervals of from 20 to 40 ft. The heel-wall should be tight; the toe-wall is pierced by ample weep-holes to relieve underpressure. The driving of sheet-piling is often impracticable on account of boulders. The foundation walls are brought up to the level selected for the bottom of the main section, and the material in the spaces between them is leveled to correspond. The solid gravity section is then laid on the whole as a foundation. By thus placing the bearing surface of the foundations deep, the danger of undercutting is minimized.

The principles of analysis used by the writer in determining a practical cross-section, for two of these dams which have been built and well tested, are simple, and can be seen by reference to Fig. 1, which was the writer's study in planning the Granite Reef Dam. The space behind the dam may always be expected to fill to the crest with river deposits, hence the provision for mud pressure. Although the cross-section is chosen for stability against overturning, using the bottom of the main section as a base, the foundation walls are monolithic with the upper parts, and contribute a heavy additional weight and frictional resistance acting against such overturning forces. As to sliding: the gridiron foundation walls enclose a very heavy mass of gravel which must be moved along with the structure or be separated from it by the parting of the concrete walls, either contingency being very strongly resisted. The underflow through the deposits beneath the dam has been found in excavations to be slow, and on account of the quantity of coarse material not easily washed, the first flood with its silt deposit makes an efficient stopper of seepage.

The upward pressure beneath the dam to be chosen for these cases is uncertain. The design of the Granite Reef Dam was made before the publication of the interesting analysis of upward pressures by G. E. P. Smith, Assoc. M. Am. Soc. C. E., and Professor H. C. Wolff, of the University of Wisconsin, contained in discussions on "Dams on Sand Foundations," by Arnold C. Koenig, Assoc. M. Am.

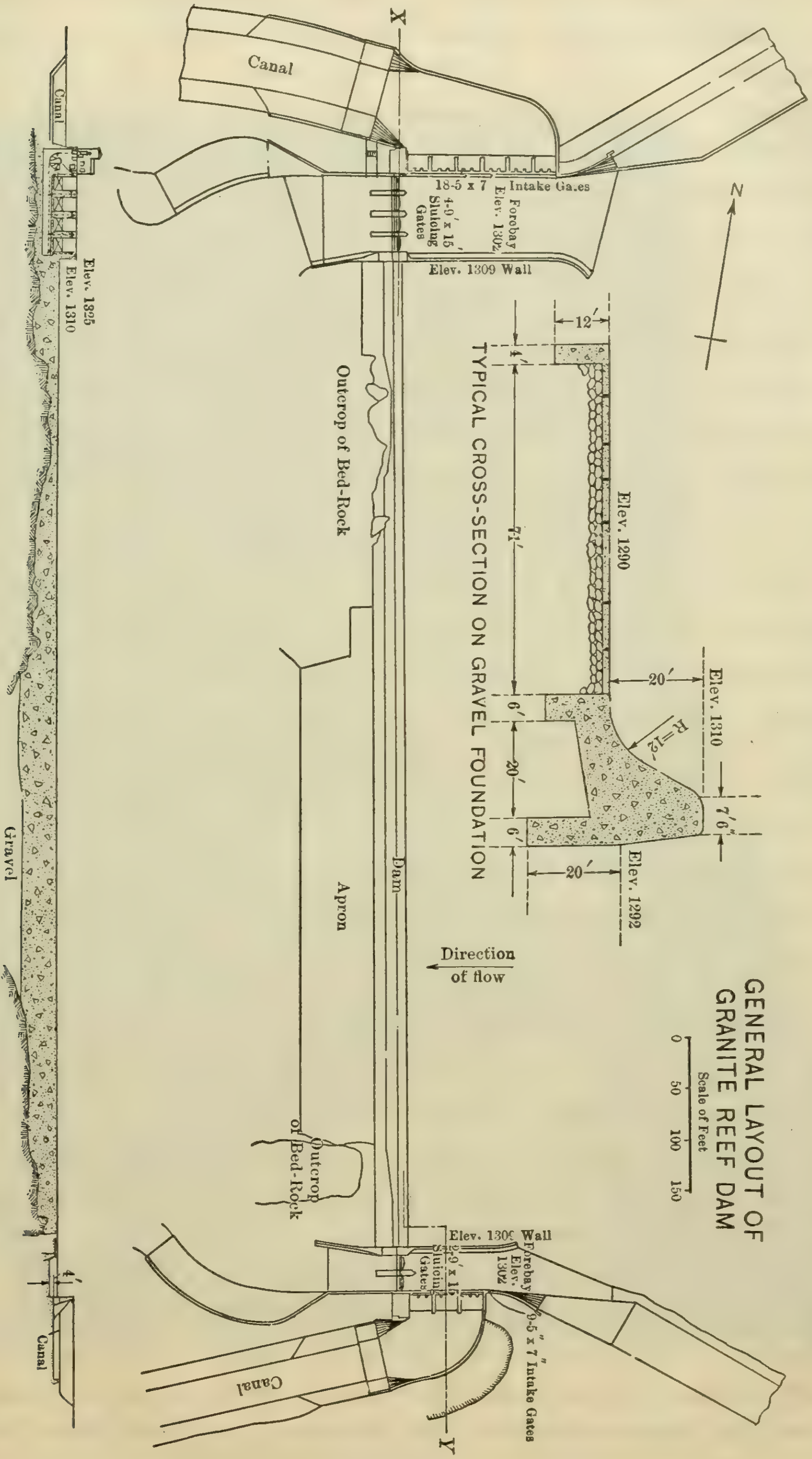

GENERAL LAYOUT OF GRANITE REEF DAM
Scale of Feet
0 50 100 150
N
X
Y
Canal
Canal
Canal
Canal
18-5 x 7 Intake Gates
Forebay Elev. 1302
4-9′ x 15′ Sluicing Gates
Elev. 1309 Wall
Elev. 1325
Elev. 1310
Outcrop of Bed-Rock
Outcrop of Bed-Rock
Dam
Apron
Direction of flow
Gravel
TYPICAL CROSS-SECTION ON GRAVEL FOUNDATION
12′
4′
71′
6′
20′
6′
20′
20′
Elev. 1290
Elev. 1310
R=12′
7′6″
Elev. 1292
Elev. 1309 Wall
Forebay Elev. 1302
2-9′ x 15′ Sluicing Gates
9-5 x 7 Intake Gates
4′

FIG. 2.

as the hydrostatic pressures under the structure are concerned, the apron with its open joints must be an area where under-pressures from above the dam and from the tail-water below are both relieved. It can be readily seen that with a tight apron of small thickness, dangerous upward pressures might be brought to bear on it. It is quite conceivable that the swiftly passing water over the open joints of the apron exerts some suction or injector action on the water which rises from beneath, thus assisting the escape of those waters and lessening their pressures.

No expansion or settlement joints were provided in the two Government dams referred to, and no evidences have appeared that they would be desirable. The dams have developed a few transverse cracks, which are no detriment, as they will silt up. There is also no sign of settlement shown by the cracks.

It is well to keep the dam and the various gates close together, usually in one continuous structure. The sand sluices must be placed with the object of disposing of sand which threatens to enter the canal intake. This sand can best be deposited in the river bed below the dam, to which place it is usually sluiced through a passage for the purpose. A good arrangement is to combine the gates and intake structures with the abutment wall at the end of the dam, and it is very desirable for this purpose to have bed-rock at these points for foundations.

Very careful attention must be paid to the apron laid along the bottom of the stream to protect the toe of the dam from being undermined. Even where a bed-rock foundation is practicable, the apron has been found to be a part of the structure very vital to its stability. Its surface should be placed at a level from 2 to 4 ft. below the natural bed of the stream. A gravel bar will then form just beyond its downstream edge which will back the water over the apron and keep a water cushion always at the toe of the dam for receiving the shock of the overflow. The width of apron sufficient to protect the toe of the dam varies according to the height and volume of the overflow and the consequent scouring energy of the water. The depth of high water above the crest will also influence the width. The apron is of concrete, but must not be connected to the dam, or in one continuous sheet. Being laid on beds of gravel and boulders near the surface, it will heave and settle more or less, and, if laid in a continuous sheet,

this action cracks it into irregular, sharp-angled slabs which are likely to be shifted and undermined by the flowing water.

A satisfactory apron was made for the Granite Reef Diversion Dam by preparing a bed of packed clean boulders in which many of the stones projected prominently. A bed of concrete, about 18 in. thick and divided into 10-ft. square slabs by vertical joints, was laid on top of this foundation. On account of the adhering boulders on the under sides of the slabs, together with their weight, they are adapted to offer great resistance to sliding. The dam raises the water level 20 ft. The apron was 75 ft. wide, down stream from the toe to the dam. Besides dividing the apron into heavy slabs of definite shape, the joints between the slabs also served to relieve the upward pressure of underflowing water during the first few months, while the river bed above was sealing itself up with silt and sand. A curtain-wall, braced with piles and waling pieces, was used to strengthen the unsupported edge of the apron. A similar apron has proved satisfactory at the Diversion Dam of the Government Power Canal on Salt River, 19 miles above the Roosevelt Storage Dam.

The rollway has a surface sufficiently outside the path of free fall to be sure that an adhering nappe is preserved. The writer does not favor a turned-up toe for the main section, as it tends to cause increased agitation of the water on the main structure. It is better to let this energy be spent on the apron and on the bar below it. This result was secured at Granite Reef, as illustrated by Fig. 3.

Gates.

The entrance of sand into the canals has been resisted by using the principle of skimming the less turbid water from near the surface. A forebay is walled off on two sides, in the form of a comparatively deep channel running directly in front of the row of intake gates; it is open at the upper end and given outlet at the lower end by large sluice-gates emptying through an opening in the dam. The intake gates draw water from this forebay at a level considerably above its bottom, and the sluice-gates, which have large capacity, draw from the very bottom of the forebay, which has a good slope toward them. In entering the forebay, the motion of the water is checked by the increased cross-section, and the heavier sand grains settle to the bottom. At intervals the sluice-gates are opened suddenly by a hydraulic

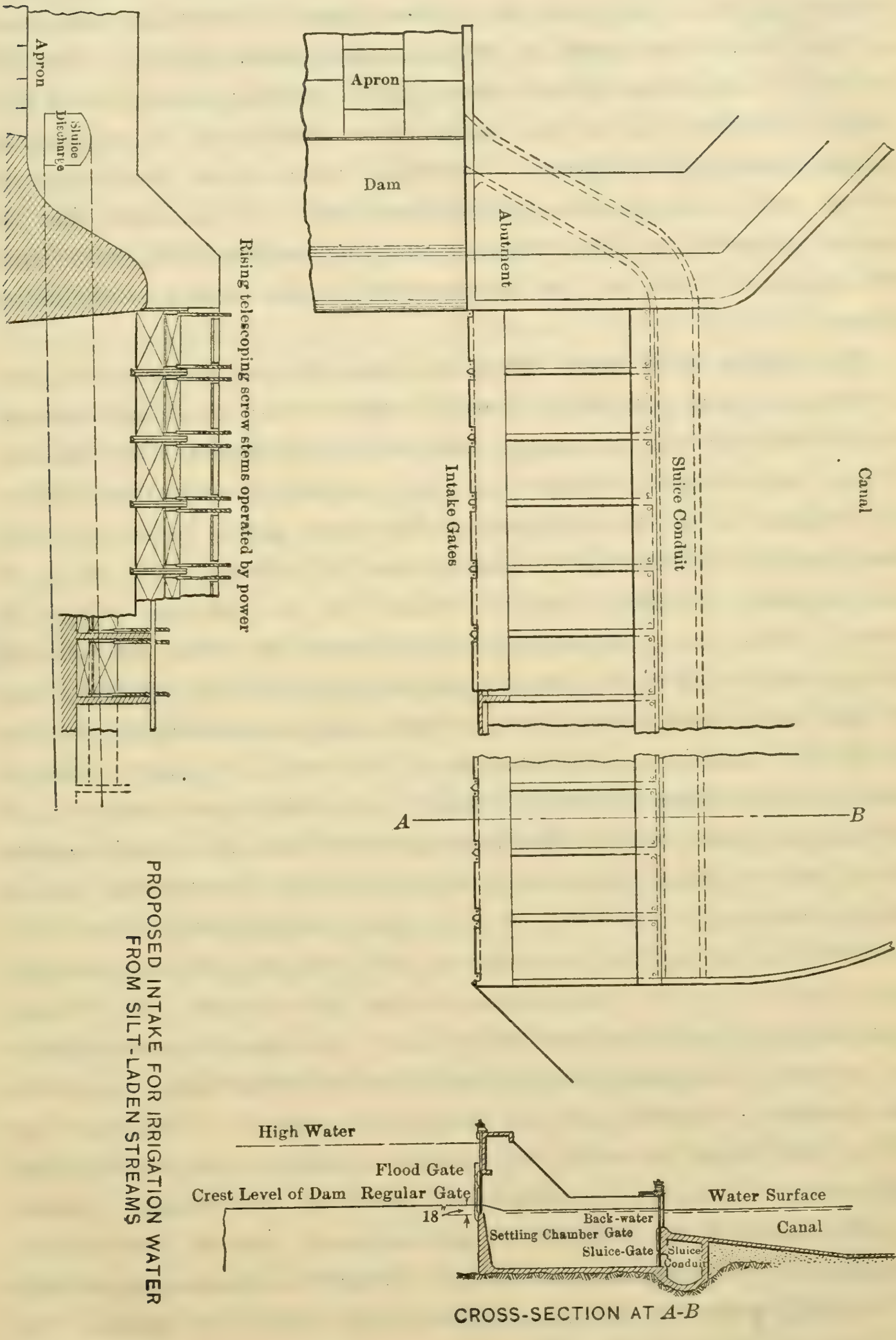


FIG. 4.

PROPOSED INTAKE FOR IRRIGATION WATER FROM SILT-LADEN STREAMS

piston, and a powerful rush of water washes out the sand. During a flood the water eddies about, and less sand is allowed to settle. With the gates placed in the usual way, if much water is drawn into the canals at these flood times, sand is carried in suspension with it. For this reason it is customary to build some kind of settling basin or sand-trap, within the first mile of the canal, where sand may be caught and sluiced back into the river. It would be better if all this separation of the sand could be done at the head-works. It is likely that the money expended on sand-traps or settling basins could, with equal advantage, be spent on the improvement of the intake sand-sluicing system.

To operate with good effect, the sand-sluicing gates must draw a current of water in front of every intake gate with enough velocity and agitation to scour thoroughly that part of the forebay. For this condition, care should be taken that the capacity to enter the forebay, as compared with the discharge of the sluice-gates, be not so large that the high velocity in sluicing occurs only close to the exit. On this account, sluicing for the benefit of the forebay cannot be done in flood time, although the sluice-gates are often left slightly open during a flood to prevent the banking of sand against them. As the water must always have enough room in entering the forebay to fill the maximum requirements of the canal, it follows that, to draw the level down and get a complete scouring at one operation, the sluice-gates must have a discharge greater than the canal requirements, unless indeed some device for controlling the entrance area to the forebay can be provided. Such a device would have to be on the outer wall of the forebay, in a very exposed position in flood season, and would add one more complication to the gate system. In the best examples now built there are rather heavy and expensive sluice-gates, on account of the large quantities of water to be handled. The writer has recently designed diversion works for a canal, to be taken out of the Gila River, in which he attempts to improve the sand-sluicing equipment. A modified arrangement based on that design is shown by Fig. 4 which he proposes as an improved type of intake works suitable for silt-laden rivers. In this design the skimming principle is carefully preserved for use in flood time when most needed. For the ordinary low-water conditions, the water is clearer, and is taken from the river over a series of wide, shallow crests, which reduce the

slope of the river bottom in the approaches to one so gentle that coarse materials will hardly be carried close to the gate entrances.

In flood times the forebay of the old type fills with gravel which is not easily sluiced out afterward. With the proposed arrangement, the lower part of the intake gate, called the "Regular Gate" is closed as soon as the flood rises 2 or 3 ft. above the crest of dam, and water for the canal is taken over the top of this gate, being controlled by the upper part, called the "Flood Gate", which closes down on the first as a sill. Gravel rolling along the river bottom during the flood passes on over the dam finding no place to enter or accumulate. Whatever gravel and sand does find its way into the intake gates is caught in the settling chambers (one for each intake gate) where it can always be sluiced under uniform conditions from the bottom, through the sluicing conduit, and back to the river below the dam. It is planned to have the upper leaf of the inside gates rise and cut off the back-water of the canal simultaneously with the opening of the lower, or sluicing leaf. These gates, however, can be worked independently, and, in case of need, the water may be shut off from the sluicing chambers entirely. It is believed that an intake of this type can be built and operated as cheaply as the other. A suitable power operating mechanism, which can be uncoupled for hand operation, can readily be designed for the proposed arrangement, as it has already been done for the old. It is also believed that sluicing the collected materials from one chamber of moderate size at a time, where a shallow stream with sharp fall can be secured to agitate and cut the deposit, will be more certain and thorough. All the sluicing gates discharge into the same waste conduit, as only one or two settling chambers need be flushed out at the same time.

The most satisfactory sluicing and intake gates yet devised are of the common rectangular sliding type, of cast iron or sheet steel, and operated by rising screw stems. They move on bronze sliding strips, and close on oak or metal sills at the bottom. Those at the Granite Reef Dam, built by the U. S. Reclamation Service,* are of thin cast iron, of arched section on the compression side, the tension being taken by steel rods at the back and placed across the bow of the cast-iron shell. They are light, strong, and durable. In this case, the sluice-gates are operated by hydraulic pressure pumps, the

* *Engineering News*, January 7th, 1909.

transmission being by heavy chains running over sheaves; they are weighted with concrete for closing by gravity. At the Granite Reef Dam, a very excellent feature of all the gates, which were designed by F. Teichman, M. Am. Soc. C. E., is that, on the pressure side, where sand, etc., is likely to bank against them, they present smooth fronts, instead of a system of deep ribs, as often built in the past.

In designing gates for these situations, the question of the proper coefficient of starting friction to use for the bronze sliding ways comes up. The writer's experience has shown that this coefficient is much larger than published authorities known to him would indicate. In order to make a reliable determination of the coefficient under actual working conditions, the writer disconnected the raising mechanism of a 5 by 7-ft. cast-iron gate, having machine-bronze sliding ways, in the power canal at Roosevelt, and in its place attached a long timber lever resting on a fulcrum made of a piece of round steel shafting held between flat steel plates. A platform for carrying weights was then suspended from a definite point on the lever arm, and the apparatus was used to weigh the starting resistance. The actual weight of this particular gate was known from the inspector's weight obtained at the time of its receipt from the manufacturer. The timber lever was weighed and its weight per linear foot assumed to be constant. A correction for the weights of the lever arms was then made.

Statement of Conditions of Test.—

Water pressure on only one side of gate;	
Size of gate opening	5 by 7 ft.
Area subjected to water pressure on the gate, measured on the center line of closing strips.	38.7 sq. ft.
Depth of water at sill of gate	8 ft. 8 in.
Head on center of pressure area	6 ft. 0 in.
Total water pressure on gate	14 500 lb.
Weight of gate	4 300 lb.
Sliding strips	Machined bronze.

Two experiments were made, as follows:

First Experiment.—To determine coefficient of starting friction after the gate had been closed tight for several weeks:

Result: Frictional resistance	9 062 lb.
Coefficient of starting friction	0.625

Second Experiment.—To determine coefficient of starting friction with the gate raised off the sill about ½ in. and water escaping under its lower edge:

Result:	Frictional resistance........................	8 985 lb.
	Coefficient of starting friction................	0.62

The turbid condition of the water, perhaps, is the chief cause of the increased size of the coefficient.

In nearly all cases it would probably pay to use gasoline engine power for operating the gates. These engines are now so common that one suitable for the power required may always be found, and speed in opening and closing is essential. They should be designed in such a way, however, that hand power may be applied in case of necessity.

Trash-racks and booms, for protecting the gates from driftwood, have not been found necessary. Most of the driftwood comes down in flood time, and then it is nearly always carried in the middle of the stream and over the dam, as it collects in the stronger current in that part of the river. If the dam is placed below, or on, a sharp bend in the stream, which is not good policy, the drift, of course, will be thrown close to the outside shore and will need to be guarded against.

Special Cases.

In Arizona there is much more land fit for irrigation than can ever be properly irrigated from the flow of the streams of that State, and because of this abundance of land, as compared with the available water in the regular flow, it is always in order to look for storage opportunities along a stream. It sometimes happens that a natural storage basin may be found near enough to the head of the farming area to make the storage dam serve the purposes of a diversion dam as well. Diversion is then a question of conduits and gates for conducting the water past the dam and into the canals. As these conduits and gates are a necessary part of the storage dam in any case, they need not be noticed in detail in this paper.

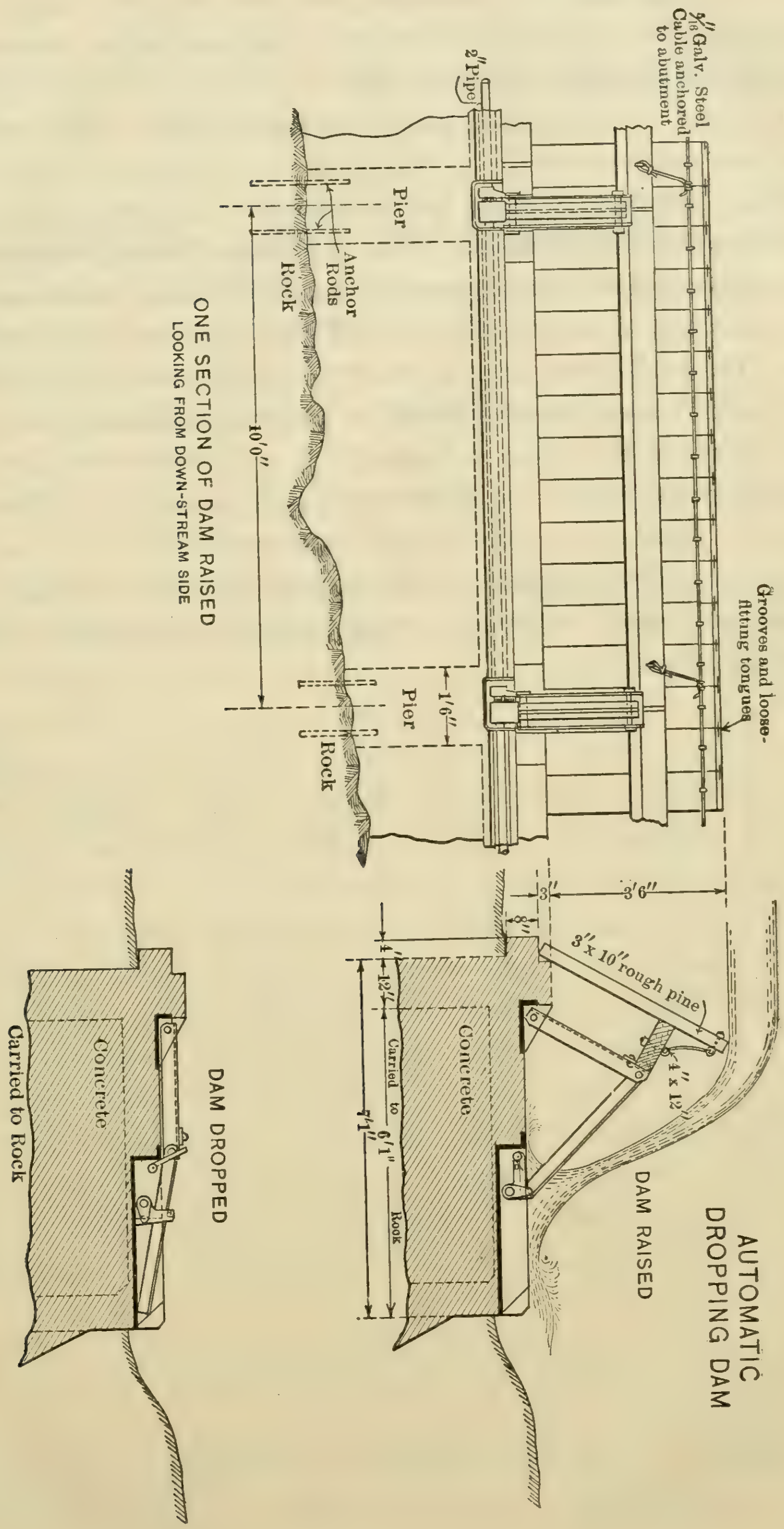
Occasionally, water has been diverted from sandy-bottomed streams of steep grade by collecting it in perforated pipes or timbered conduits buried in the stream bed and brought to the surface by extending at a gentle grade far enough down stream to gain the necessary elevation.

Heavy floods will pass over an intake of this kind without damage, and a constant small flow can sometimes be obtained from the ground where the stream is ordinarily dry at the surface.

Fig. 5 illustrates a plan recently devised by the writer for a case on the Gila River where the area of the land under the canal would not justify the expense of a permanent structure extending entirely across the river bottom, which at this place is wide. The canal company has heretofore maintained at its head-works a brush and earth dam which has been carried away by small rises of water several times in a season. The great loss from this condition, as mentioned previously, is to growing crops which are deprived of water. The main object of the present plan is to provide an escape for the small summer rises in the river (say from 2 000 to 10 000 sec-ft.), which are of short duration, without at the same time losing the head necessary to keep the canal running. As the flow during winter floods, and an occasional summer flood, is very much greater than the foregoing figures, the dam, which raises the water only 3½ ft., is designed to go out automatically when the danger level is reached, thus largely increasing the discharge area and leaving no obstruction in the channel above the concrete foundations, which rise only to the natural bottom level of the river. When a flood has thrown down the dam, it will be comparatively easy to set up the steel frames again and put in place the planks which make the weir. The concrete foundation is on a bed-rock footing. It is expected that the earth dike, which closes the gap from the automatic dam to the farther shore, will be broken through by occasional extreme floods. It should be planted with willows or cottonwoods and Bermuda grass to help bind it together.

For the purpose of springing the latch which holds up the first 10-ft. section of the dam, it is proposed to have a protected well, connected with the river water, in the masonry of the abutment at the shore end, in which is enclosed a heavy float operating the necessary lever. The fall of the first frame is designed to trip the latch of the next, and so on until the fall of the dam is complete.

The individual planks of the weir are attached to a wire cable, the end of which is free at the point where the sections are first released, the other end being anchored to the opposite end pier. This precaution is taken to save as many pieces of this timber as possible



5/16" Galv. Steel Cable anchored to abutment
2" Pipe
Pier
Anchor Rods
Rock
ONE SECTION OF DAM RAISED
LOOKING FROM DOWN-STREAM SIDE
10'0"
1'6"
Pier
Rock
Grooves and loose-fitting tongues
3"
3'6"
8"
3" x 10" rough pine
4" x 12"
4"
12"
Concrete
Carried to 6'1" Rock
7'1"
DAM RAISED
AUTOMATIC DROPPING DAM
DAM DROPPED
Concrete
Carried to Rock

Fig. 5.

for use in replacing the dam after the flood has run down. It is expected that the string of planks will float and swing over to shore, where the lumber may be recovered.

It should be understood that the types of works mentioned in this paper are not advanced as suitable for all conditions to be found in Arizona. Every case must be considered in relation to its peculiar conditions and requirements. The Colorado River presents a series of conditions, due to its enormous volume of flood discharge, and to the conditions of its bed, that demand particular and unclassified solutions. For such water-sheds as those of the Gila and Salt Rivers, however, and for many other streams in the arid Southwest, it is believed the contents of this paper will apply. It will be observed that the best examples noted herein are structures put in by the U. S. Reclamation Service. These have been planned and constructed with an amount of care not often equalled in the case of structures built by private parties, and much valuable investigation and observation have been devoted to them.

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PAPERS AND DISCUSSIONS

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STEEL STRESSES IN FLAT SLABS.*

By Dr. H. T. Eddy.†

The question of the percentage of reinforcement in flat slabs is important economically as well as theoretically. It is, moreover, a question respecting which there seems to be some misconception as to the principles which should be applied. Shall the concrete and steel be balanced as perfectly as possible; or, if not, which shall be made the stronger? The uninformed have usually insisted on over-reinforcement, so that the concrete would be the weaker. If it is possible to determine the actual stresses in the steel with precision, there is no doubt that slabs should be designed so that the steel will give way first. Concrete is an unreliable and fragile material compared with steel. Under-reinforcement, however, will make security depend on the steel, which has known properties capable of precise computation. When the steel begins to yield, that action will cause the concrete to fracture to some extent, but the structure will exhibit toughness, and will not be subject to sudden collapse. In case of over-reinforcement, where the concrete is crushed and gives way first, the structure is in danger of much more sudden and unforeseen failure.

That these conclusions coincide with those of economic design appears from the following consideration of the stresses in steel and

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

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concrete, as given by Turneaure and Maurer's equations for the equal resisting moments of steel and concrete per unit of width of slab regarded as a beam, namely,

$$M_s = f_s\, Ajd = f_s\, pjd^2, \text{ and } M_c = \frac{1}{2} f_c\, kjd^2,$$

in which $A = pd$ is the cross-section of the reinforcement per unit of width of slab, $kd =$ the depth of the neutral axis, and $jd =$ the depth of the steel below the center of compression of the concrete. As these moments are equal, the ratio of the stresses in steel and concrete is,

$$\frac{f_s}{f_c} = \frac{k}{2\,p}.$$

in which p is the steel ratio of reinforcement. Using Turneaure and Maurer's tabular values of k, in case $n = \frac{E_s}{E_c} = 15$, and assuming that $f_c = 500$ lb. per sq. in., and $f_s = 16\,000$ lb. per sq. in., the values of stresses in steel and concrete, respectively, shown in Table 1, are found.

TABLE 1.

$n = 15$.		$\frac{f_s}{f_c} = \frac{k}{2\,p}$.	For $f_c = 500$ $f_s = \frac{250\,k}{p}$.	For $f_s = 16\,000$ $f_c = \frac{32\,000\,p}{k}$.
p.	k.			
0.0025	0.25	50	25 000	320
0 005	0.32	32	16 000	500
0.010	0.42	21	10 500	762
0.015	0.48	16	8 000	1 000
0.020	0.525	13	6 550	1 220

Table 1 shows how small the percentage of reinforcement should be (less than one-half of 1%) in order that, in case of under-reinforcement, the steel should be used with reasonable economy; also, in case of over-reinforcement, how great the stresses in the concrete become in order to develop the strength of the steel. Every consideration of correct design, safety, and economy seems to demand a very low percentage of steel, verging on under-reinforcement, rather than on any attempt at over-reinforcement.

If these principles are adopted as controlling the design of flat slabs, the all-important and crucial question is that of unit stresses in the steel. The writer hopes that the following discussions of tests

for actual stresses in several large buildings, as compared with computed theoretical stresses, may assist in supplying a satisfactory basis for computation and design such as has not been available hitherto.

The Northwestern Glass Company Building.

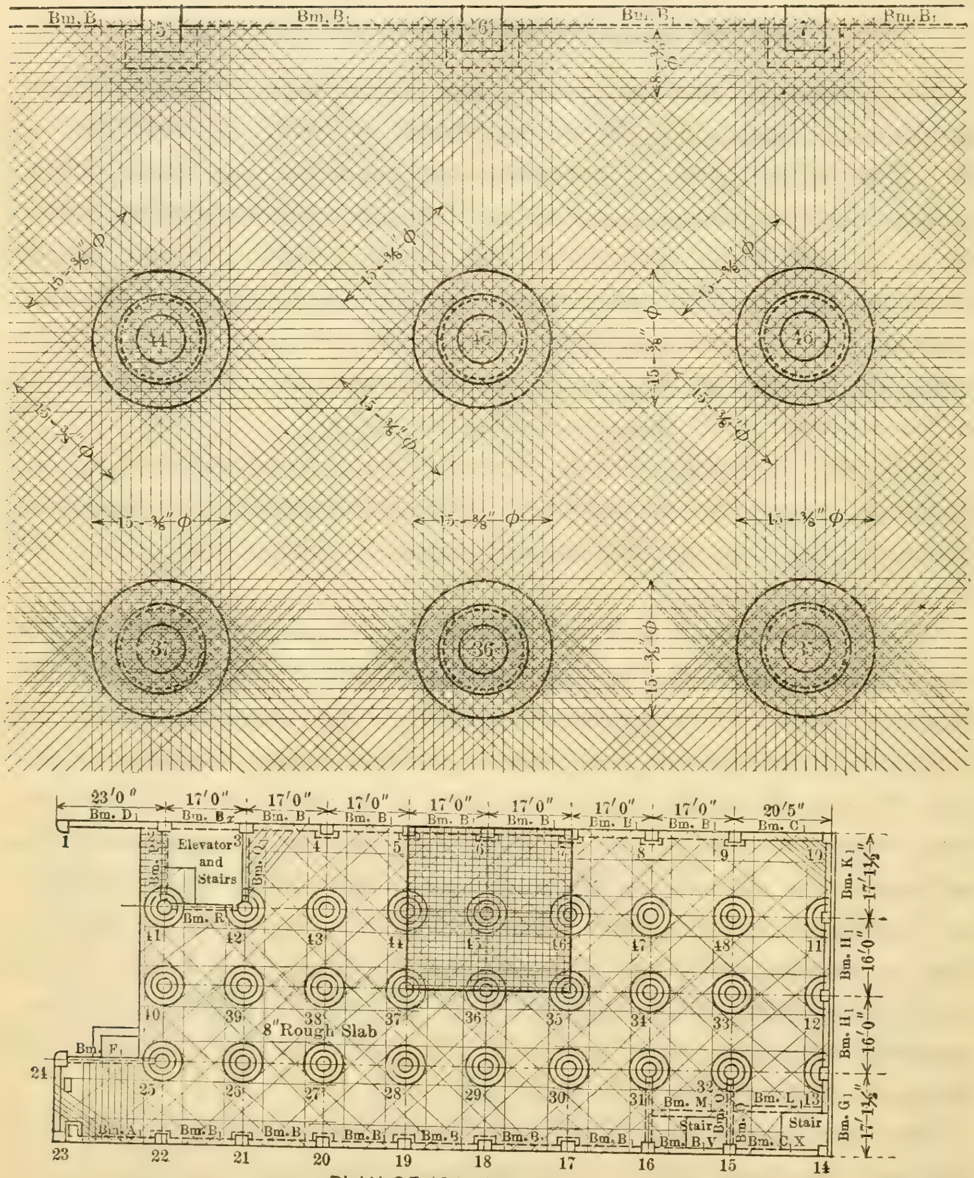
The mushroom, flat slab floor in the Northwestern Glass Company Building, in Minneapolis, Minn., was tested by Mr. F. R. McMillan, on May 12th to 24th, 1913. The pressure for immediate occupancy of the building by the owners was such as to compel the completion of the test at an undesirably early date after the slab, which had been frozen all winter, had sweated out, and had had an opportunity to commence to become cured. It is estimated that the condition of the slab at the beginning of the test was perhaps such as would be expected at the end of a period of 45 days favorable for curing.

Appendix A contains the particulars of the test conducted by Mr. McMillan, whose results are used in this discussion. The equations applied in the computations may be found in the writer's book, "The Theory of Flat Slabs."*

The maximum load on Panel D amounted approximately to 185 000 lb. This was placed on the four quarters of the panel, with open passageways, 1.5 ft. wide, across the middle of the panel each way, and with open spaces around the columns at each corner arranged so that only 181.5 sq. ft. of the total panel area of 16 by 17 ft. = 272 sq. ft., were actually covered by the loading. With such an arrangement of the loading it was difficult to determine just what should be assumed as the equivalent uniform load. The difficulty is increased by the fact that, no matter what load is placed on the mushroom heads, it will have little effect in increasing either stresses or deflections. Consequently, attention must be given principally to the equivalence of the loading not located over the heads.

The open passageways decreased the effectiveness of the actual loading, and the open spaces over the heads increased its effectiveness in a very complicated manner when one also considers their stiffness relative to the remainder of the panel area. It will be evident, however, that, on the whole, the loading as actually applied, was more effective in producing deflections and stresses than if the total actual

* The numbers of the equations in this paper are the same as those in "The Theory of Flat Slabs."



PLAN OF 1ST FLOOR

REINFORCING PLAN OF TEST PANELS
TEST OF
NORTHWESTERN GLASS CO. BUILDING.
FIG. 1.

load had been distributed uniformly over the panel. This takes into account the fact that the load, almost exclusively, was on the more flexible portion of the panel. In default of any exact analysis, it has been assumed that a total panel load of $W = 200\,000$ lb. uniformly dis-

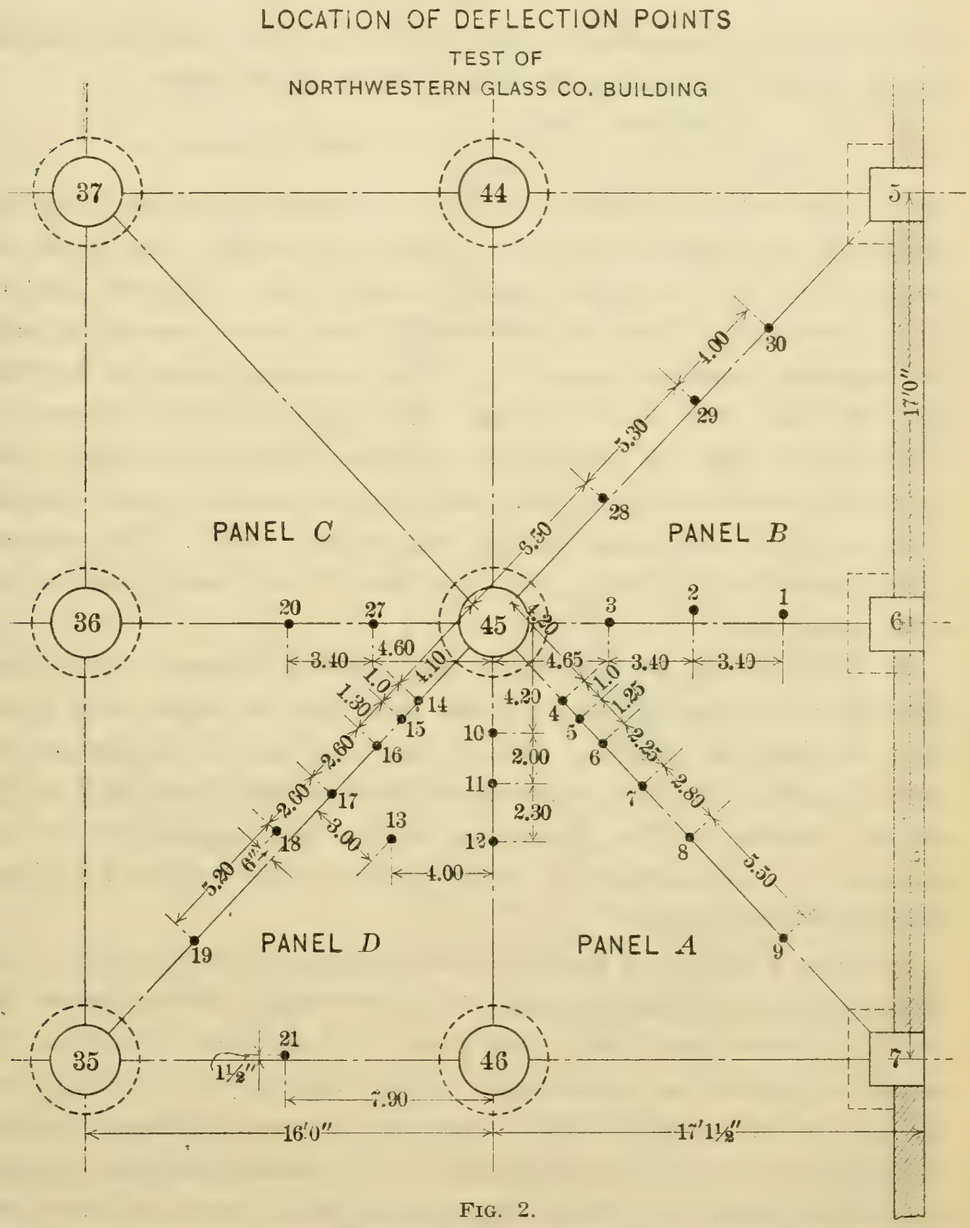


FIG. 2.

tributed would cause deflections and stresses at least as great as those due to the actual load of 185 000 lb.

This value of W will be used in the computations as equivalent to the actual load, although it is evident that, in computing deflections,

a value of W, somewhat different from that needed in computing stresses, might be required.

The writer has proposed the following equation

$$f_s = \frac{W\,L}{175\,d_1\,A} \qquad \qquad (34)$$

as giving the limiting unit stress in the rods of the side belts. In the present case this gives, for the rods of the short side belts,

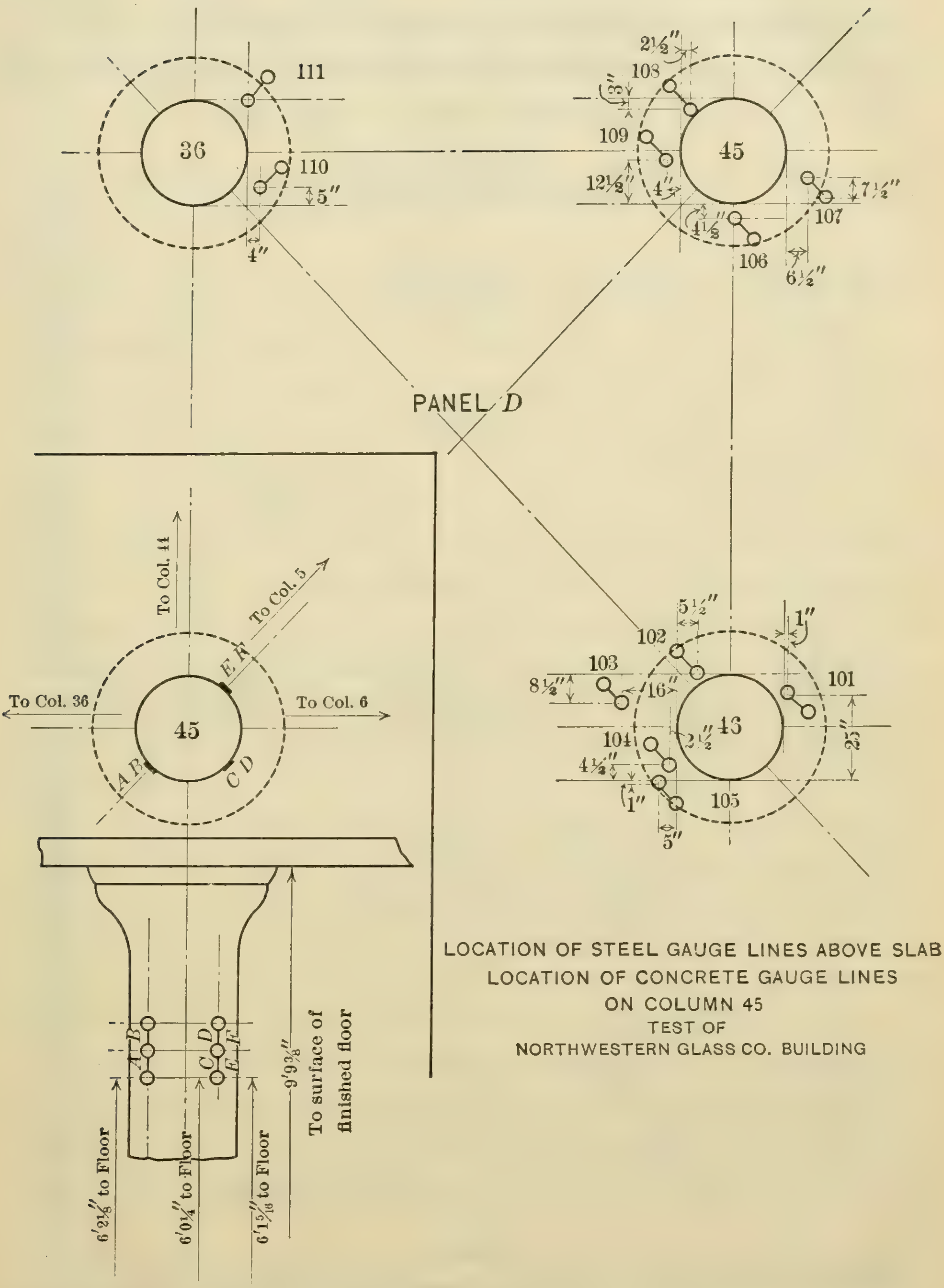
$$f_s = \frac{200\,000 \times 16 \times 12}{175 \times 7.31 \times 1.6567} = 18\,000 \text{ lb. per sq. in.}$$

Two of the observed readings, on Fig. 4, namely, those at 3 and 10, amounted to 17 000 lb. per sq. in., and it is probable that if all the panels of the slab had been loaded at once with a uniform load of $W = 200\,000$, all these side rods would have shown nearly or quite the foregoing computed value of f_s. That this is so seems to be clear from the fact that these readings were taken in belts adjacent to panels which were not loaded, and that the stress at 10 under Load 3, when the total load was placed nearly equally on four panels instead of on two, was not much smaller than under Load 5. The changes in the stresses at 7, 8, and 9, between Load 3 and Load 5, lead to the same conclusions.

It will be seen, by comparing the observed stresses in the short side belt with those in the long side belt, that the latter were somewhat less than the former. This is the only kind of divergence between Equation (34) and experimental results which has come to the writer's attention. The divergence in this case appears to be due primarily to the nearness of these panels to the wall and to their position relative thereto.

Suppose a slab to be loaded in such a manner that the entire load was placed in continuous narrow strips, extending entirely across the middle of each panel, in a line parallel to the wall. These strips might be regarded as concentrated central loads on a wide beam having one end at the wall, with a depression or trough between the wall and the first row of columns parallel to it, another between the first and second rows, etc. These depressions would be much more pronounced than any crossing them perpendicularly to the wall.

The wall and Load 5 evidently had an effect of this kind, which caused the stresses in the short side belts perpendicular to the wall to exceed those in the long side belts.



111
110
5″
4″
36
45
2½″
108
3″
109
12½″
4″
4½″
106
107
7½″
6½″
PANEL D
To Col. 44
To Col. 5
To Col. 36
To Col. 6
E F
A B
C D
5½″
1″
102
103
8½″
16″
101
104
2½″
43
25″
4½″
1″
105
5″
6′2⅛″ to Floor
6′0¼″ to Floor
6′1$\frac{5}{16}$″ to Floor
9′9⅜″
To surface of finished floor
LOCATION OF STEEL GAUGE LINES ABOVE SLAB
LOCATION OF CONCRETE GAUGE LINES
ON COLUMN 45
TEST OF
NORTHWESTERN GLASS CO. BUILDING

Fig. 3.

Indeed, slab action in general may be described partly as the attempted mechanical superposition of one set of parallel depressions and elevations on another set of similar corrugations at right angles to them. Such sets mutually support each other and give rise to slab

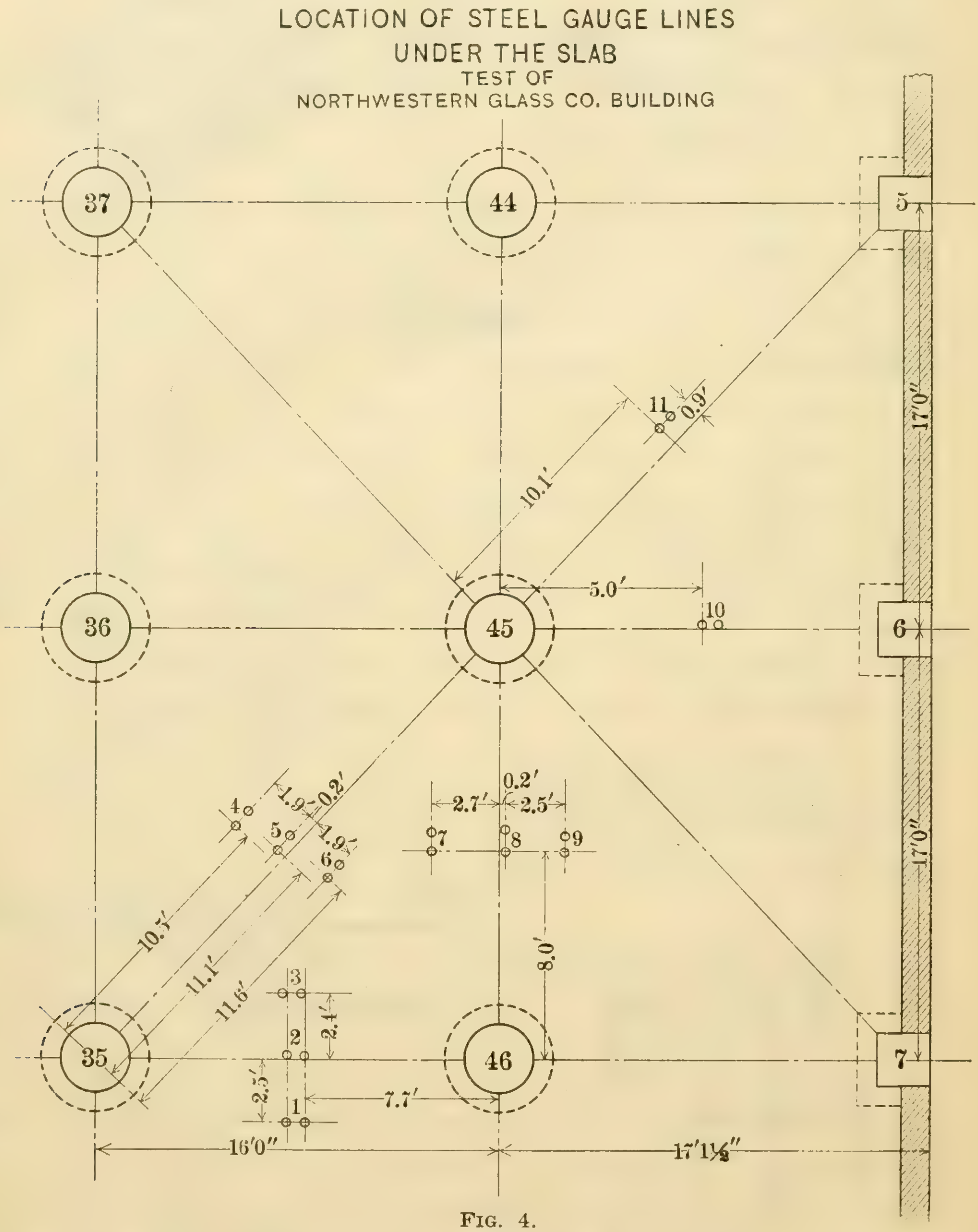


FIG. 4.

action, but any action which interferes with and disturbs the regularity of this superposition needs consideration. A wall support is such an interference, for it completely destroys the regular sequence of depressions and elevations at the end of one set and thus intensifies

those in the other set. The case just discussed is evidently of this character.

The writer has shown* that the stresses to be expected in the diagonals at the center of the panels are somewhat less than those in the middle of the side belts when all the panels of a slab are equally loaded.

By Equation (52) the unit stress in the middle rod of the diagonal at the center of the panel is

$$f_s = \frac{C_1 \, W \, L_1}{256 \, j \, d_2 \, A_1} = \frac{200\,000 \times 204}{256 \times 0.89 \times 6.94 \times 1.6567} = 15\,570 \text{ lb. per sq. in.}$$

The observed value at 5 in Panel D was 14 200 lb., which was larger than that in Rods 4 and 6 on each side of it, as required by theory. The stress at 11 in the wall panel, C, was larger, and amounted to 20 500 lb., which probably was due to the cantilever action at the wall being less than that exerted by the usual column heads.

Consider now the stress in the belt rods near the edge of the cap. In order to do this it is necessary to treat them somewhat more in detail than has been done in the writer's book, where the entire right section of diagonal and side belts has been regarded as unaffected by the mass of the cap, which forms a large boss, integral with the slab, and, as far as it extends, renders the belt much less extensible and compressible than it would be without it. The edges of each belt are so far from the edge of the cap as to permit of the assumption that the rods at the edge have the same stresses as they would were the cap not present; but those rods which are tangent to the cap will have their elongations and stresses reduced thereby, and the rods beside the cap and intermediate between these and the edges of the belts will have their stresses reduced in proportion to their proximity to the cap.

The case is different, however, with rods which cross the edge of the cap nearly perpendicularly. Although that part of the length of the rod which is inside the cap has its elongation prevented by the mass of the cap, the part outside must have its elongation correspondingly increased to compensate for this loss, and, on the whole, be equal to that of the rods beside it. The stresses in the middle rods of each belt, consequently, are increased abnormally for this reason just as it leaves the cap. Instead of attempting to determine this increase by

* "The Theory of Flat Slabs."

some intricate investigation, it will simply be assumed that the stress at this point in the middle rod does not exceed that in the outside rod of the belt at a point opposite the center of the cap. This must be very nearly the fact, and the stress in the outside rod at this point, as affected by the size of the cap, will now be considered.

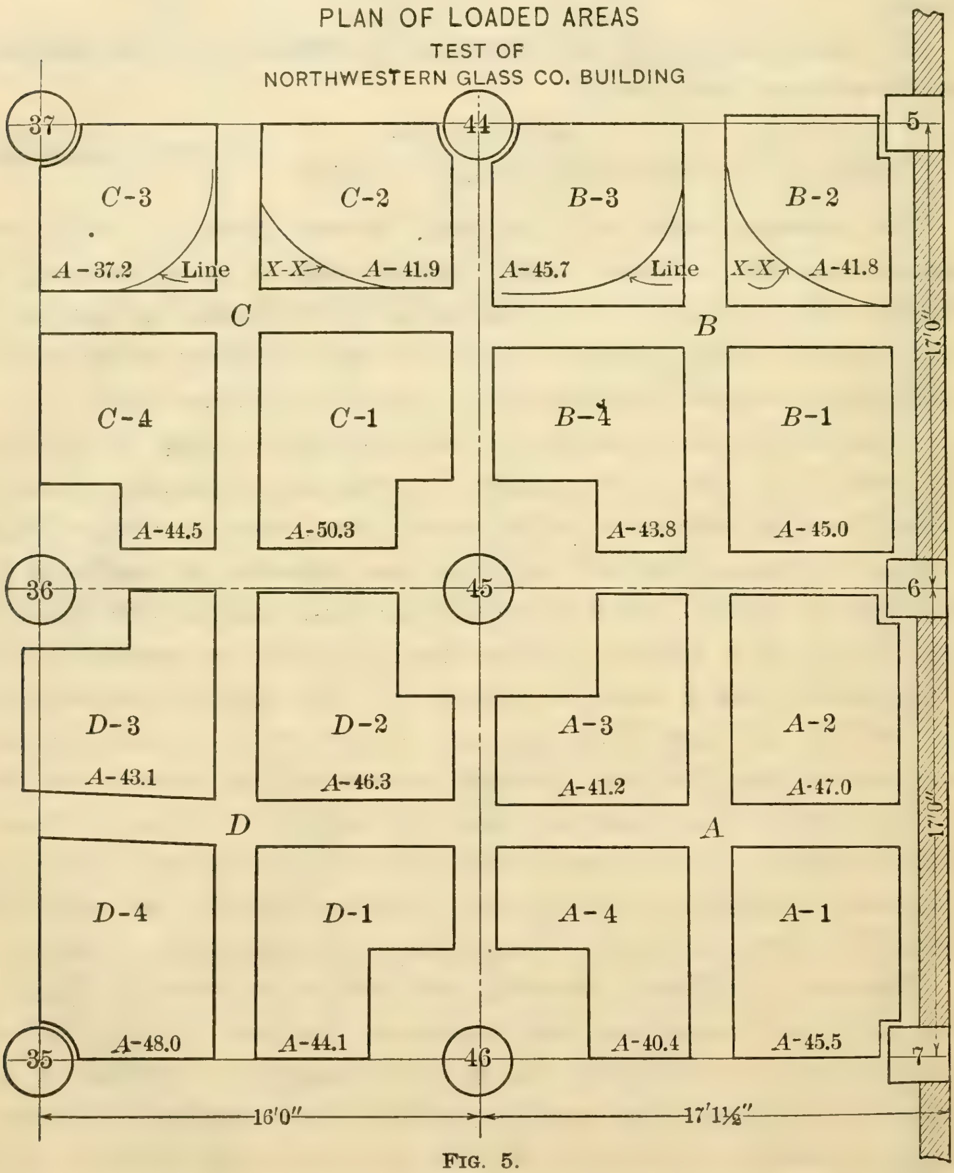
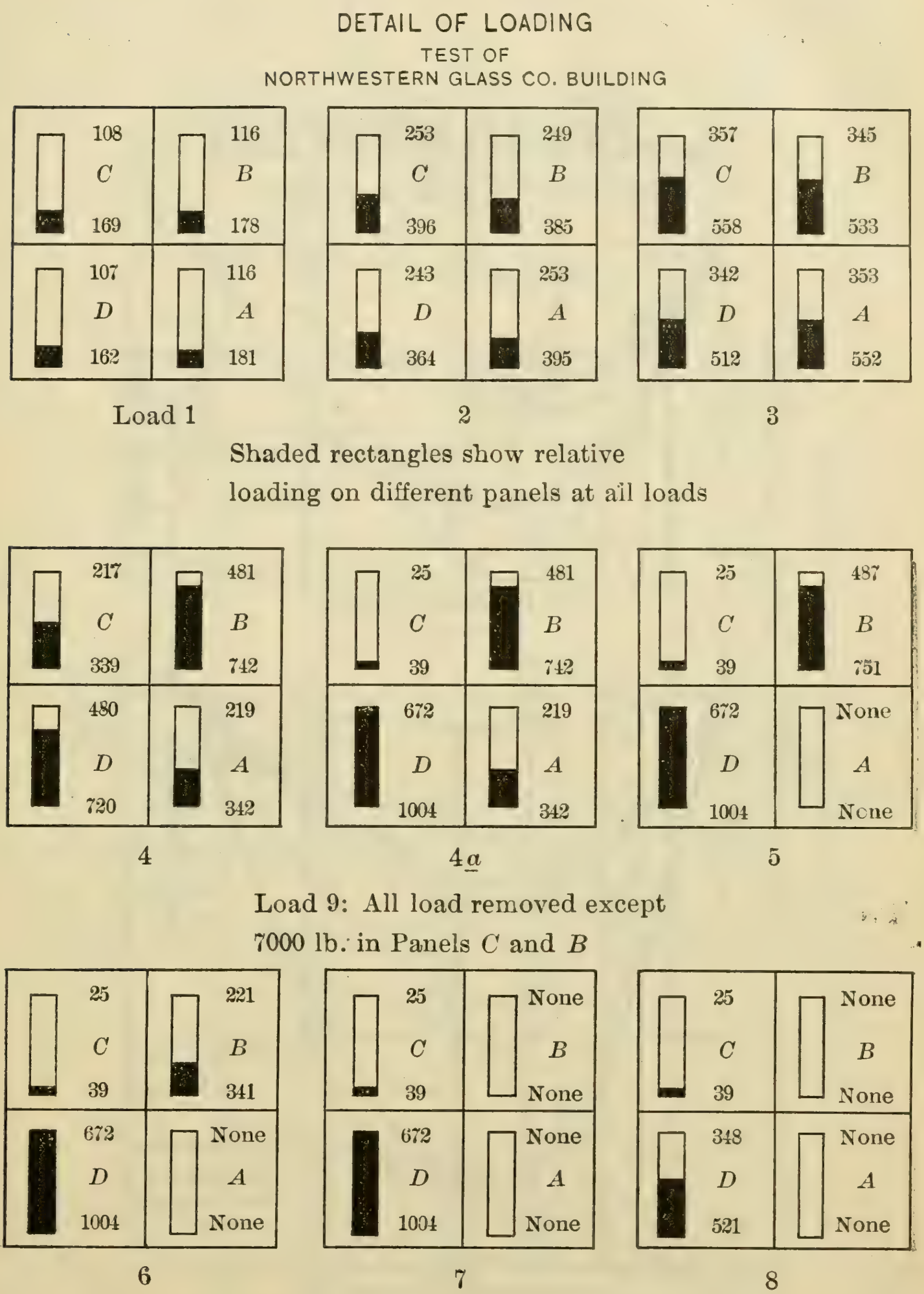


FIG. 5.

As the cap is integral with the slab and may be taken as horizontal at the edge of the cap instead of at the center of the column, as has been assumed in the writer's equations, the position of the points of inflection or contraflexure of the rods will be nearer the center of the



Upper figures in each panel show load
per square foot over the whole panel area and
lower figures over the actual loaded area.

Fig. 6.

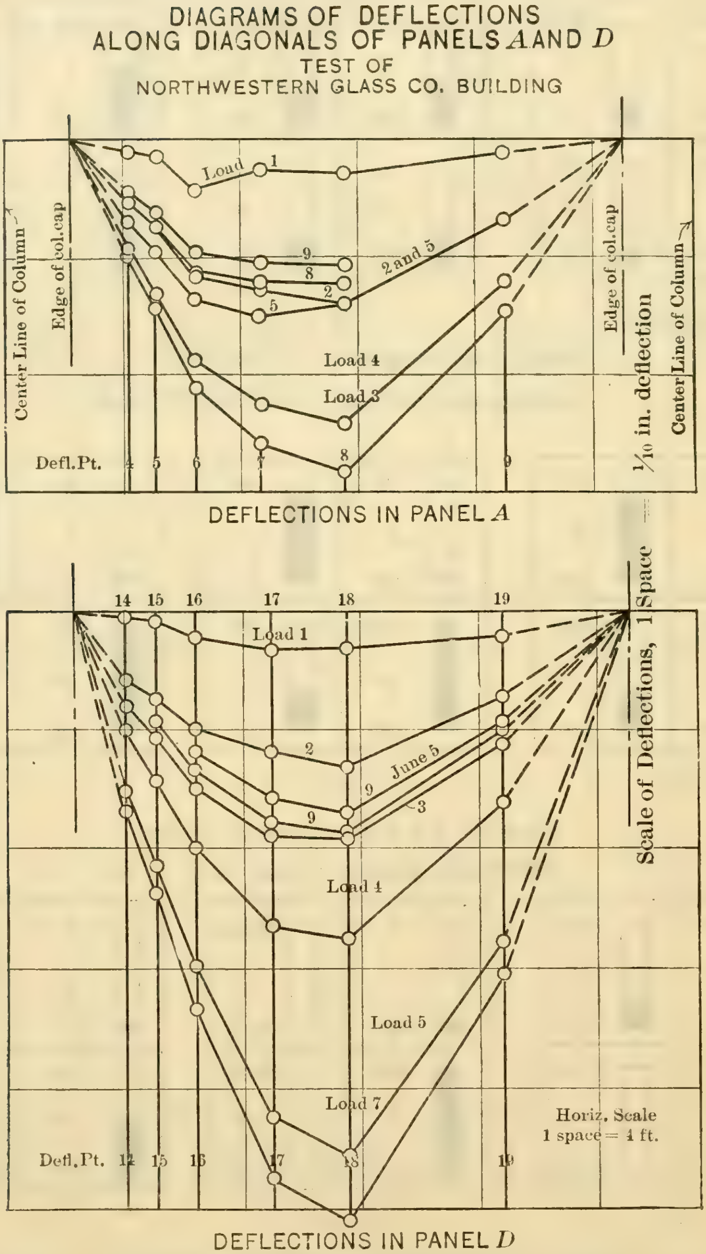

DIAGRAMS OF DEFLECTIONS
ALONG DIAGONALS OF PANELS A AND D
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Center Line of Column
Edge of col.cap
Load 1
9
8
2
5
2 and 5
Load 4
Load 3
Defl.Pt. 4 5 6 7 8 9
Edge of col.cap
Center Line of Column
1/10 in. deflection
DEFLECTIONS IN PANEL A
Scale of Deflections, 1 Space
14 15 16 17 18 19
Load 1
2
June 5
9
3
9
Load 4
Load 5
Load 7
Horiz. Scale
1 space = 4 ft.
Defl.Pt. 14 15 16 17 18 19
DEFLECTIONS IN PANEL D

FIG. 7.

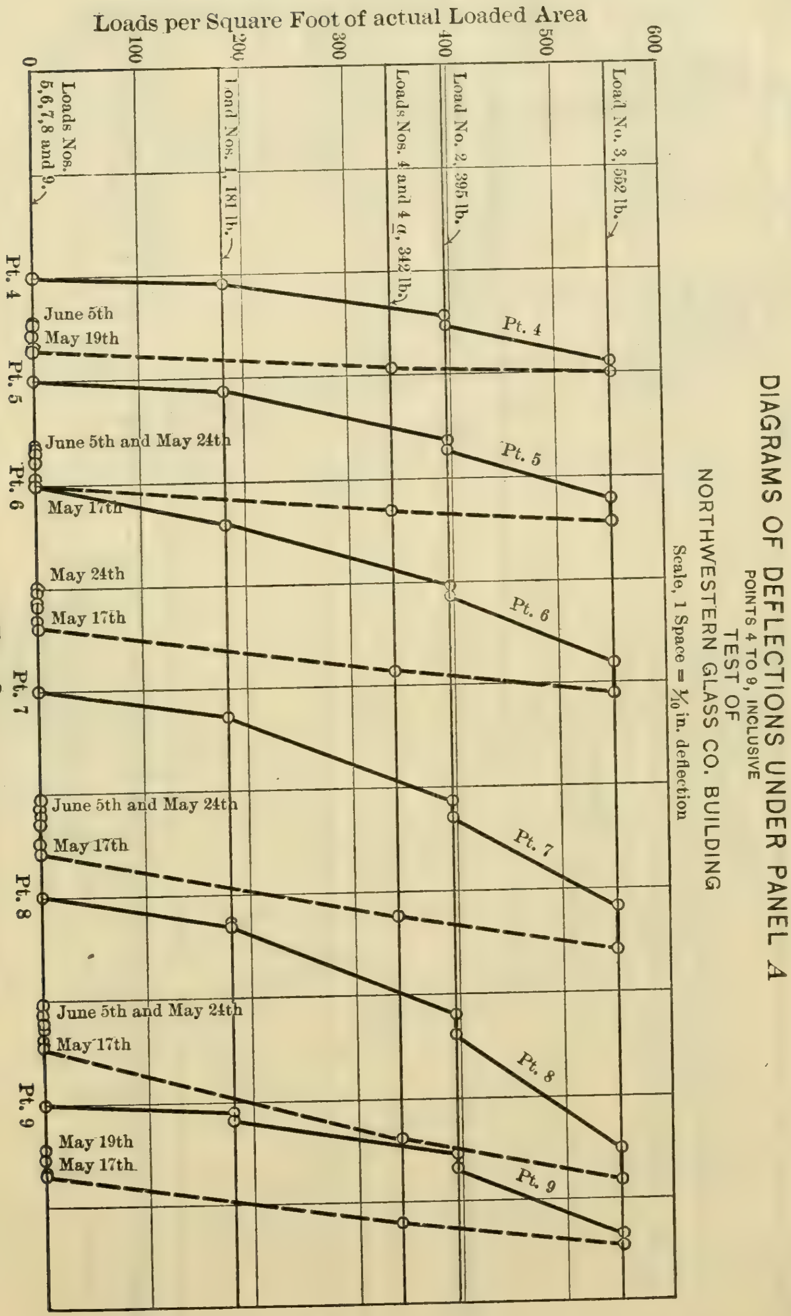

DIAGRAMS OF DEFLECTIONS UNDER PANEL A
POINTS 4 TO 9, INCLUSIVE
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 Space = 1/10 in. deflection
Loads per Square Foot of actual Loaded Area
0
100
200
300
400
500
600
Loads Nos. 5,6,7,8 and 9.
Load Nos. 1, 181 lb.
Loads Nos. 4 and 4 a, 342 lb.
Load No. 2, 395 lb.
Load No. 3, 552 lb.
Pt. 4
June 5th
May 19th
Pt. 5
June 5th and May 24th
Pt. 6
May 17th
May 24th
May 17th
Pt. 7
June 5th and May 24th
May 17th
Pt. 8
June 5th and May 24th
May 17th
Pt. 9
May 19th
May 17th
Pt. 4
Pt. 5
Pt. 6
Pt. 7
Pt. 8
Pt. 9

FIG. 8.

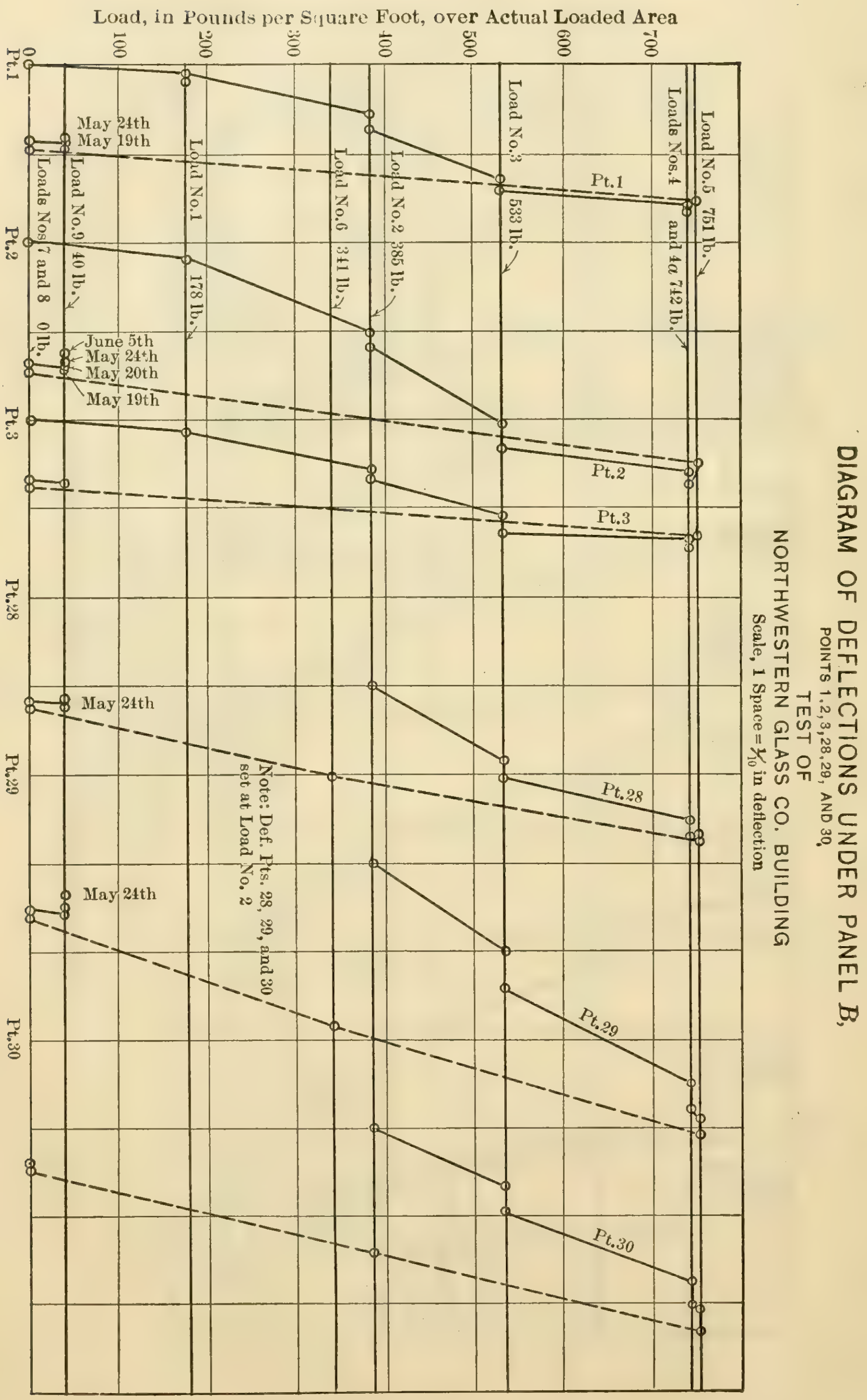


DIAGRAM OF DEFLECTIONS UNDER PANEL B,
POINTS 1, 2, 3, 28, 29, AND 30,
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 Space = 1/10 in deflection
Load, in Pounds per Square Foot, over Actual Loaded Area
0
100
200
300
400
500
600
700
Load No.5 751 lb.
Loads Nos.4 and 4a 742 lb.
Load No.3 533 lb.
Load No.2 385 lb.
Load No.6 341 lb.
Load No.1 178 lb.
Load No.9 40 lb.
Loads Nos. 7 and 8 0 lb.
May 24th
May 19th
June 5th
May 24th
May 20th
May 19th
May 24th
May 24th
Pt.1
Pt.2
Pt.3
Pt.28
Pt.29
Pt.30
Note: Def. Pts. 28, 29, and 30 set at Load No. 2

FIG. 9.

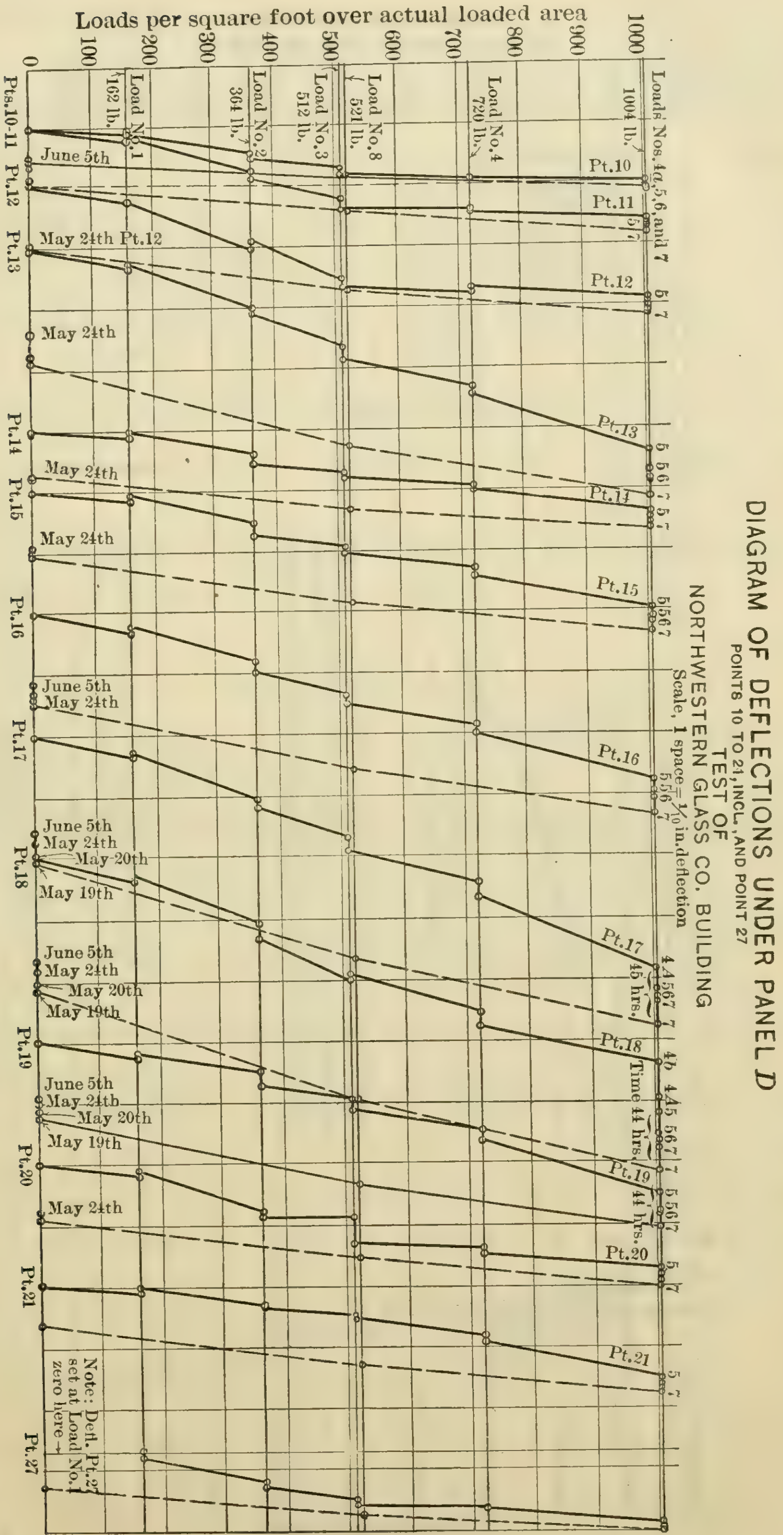
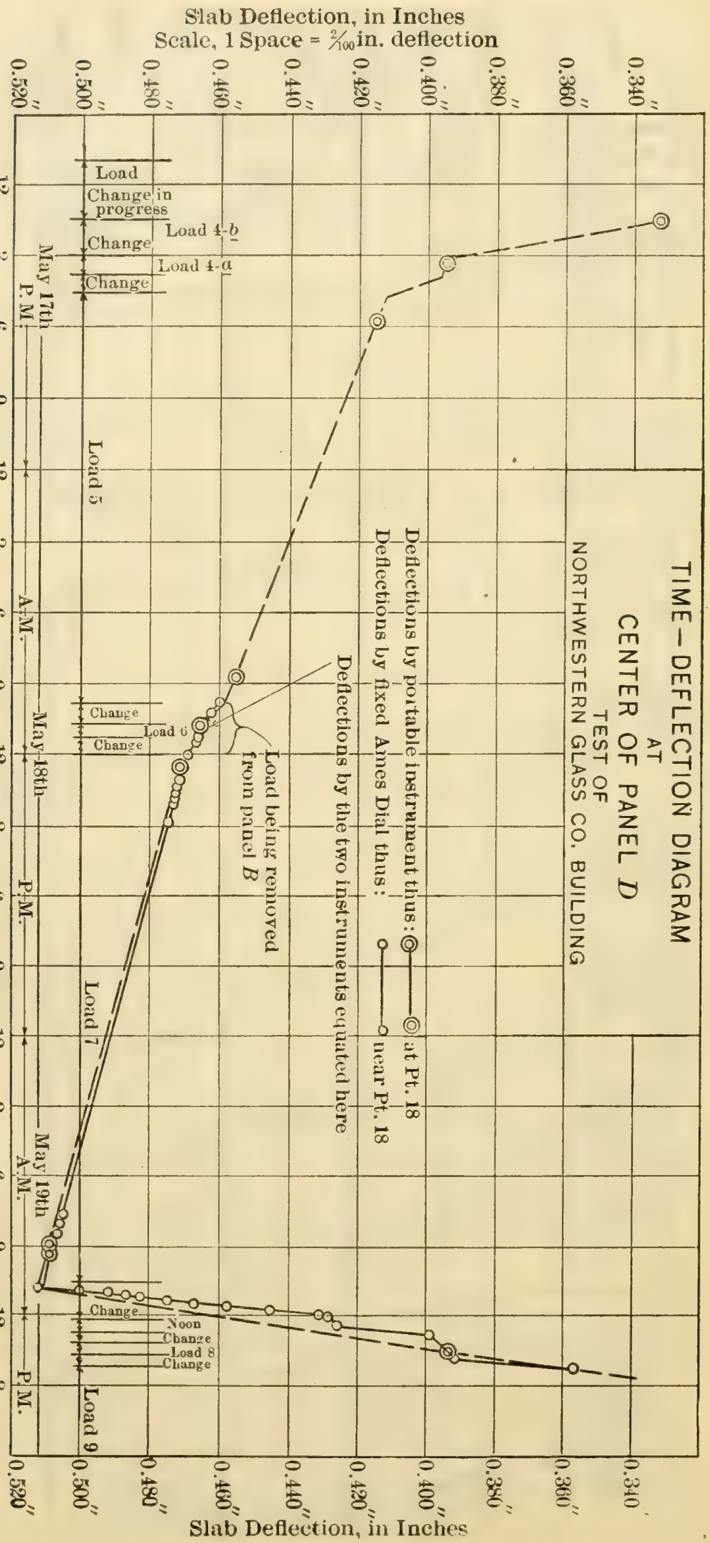


DIAGRAM OF DEFLECTIONS UNDER PANEL D
POINTS 10 TO 21, INCL., AND POINT 27
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Scale, 1 space = 1/10 in. deflection
Loads per square foot over actual loaded area
0
100
200
300
400
500
600
700
800
900
1000
Load No. 1
162 lb.
Load No. 2
364 lb.
Load No. 3
512 lb.
Load No. 8
521 lb.
Load No. 4
720 lb.
Loads Nos. 4a, 5, 6, and 7
1004 lb.
Pts. 10-11
Pt. 12
Pt. 13
Pt. 14
Pt. 15
Pt. 16
Pt. 17
Pt. 18
Pt. 19
Pt. 20
Pt. 21
Pt. 27
June 5th
May 24th
May 20th
May 19th
Pt. 10
Pt. 11
Pt. 12
Pt. 13
Pt. 14
Pt. 15
Pt. 16
Pt. 17
Pt. 18
Pt. 19
Pt. 20
Pt. 21
45 hrs.
Time 44 hrs.
44 hrs.
Note: Defl. Pt. 27 set at Load No. 1 zero here →

FIG. 10.



TIME—DEFLECTION DIAGRAM
AT
CENTER OF PANEL D
TEST OF
NORTHWESTERN GLASS CO. BUILDING
Deflections by portable instrument thus: at Pt. 18
Deflections by fixed Ames Dial thus: near Pt. 18
Deflections by the two instruments equated here
Load being removed from panel B
Slab Deflection, in Inches
Scale, 1 Space = 2/100 in. deflection
0.520″
0.500″
0.480″
0.460″
0.440″
0.420″
0.400″
0.380″
0.360″
0.340″
Load
Change in progress
Load 4-b
Change
Load 4-a
Change
Load 5
Change
Load 6
Change
Load 7
Change
Noon
Change
Load 8
Change
Load 9
May 17th P.M.
A.M.
May 18th
P.M.
May 19th A.M.
P.M.
12
3
6
9
Slab Deflection, in Inches

FIG. 11.

span than has been assumed in those equations, when account is taken of the size of the cap. In Equation (42), a stress equation, the position of the points of inflection of the side belts is determined to be at distances $\pm \frac{a}{\sqrt{3}}$ and $\pm \frac{b}{\sqrt{3}}$ from the middle of the span, because the belts are horizontal at the middle of the span and over the column centers. Now if, instead, the belt is horizontal at the middle, and at the edge of the cap, which, for convenience, is assumed to be square, the points of inflection will be at a distance from the mid-span of one-half the clear span between the caps multiplied by $\frac{1}{3} \sqrt{3} = 0.577$. Subsequently, this will be found to be in agreement with experimental determinations.

Instead of Equation (42), we shall have, for the limiting stress in the side belt rods opposite the column center

$$f_s = \frac{W L_1 (L_1 + L_2)}{800\, d_3\, A_1\, L_2} \left[\frac{3\, L_1^2}{B_1^2} - 1\right] \ldots\ldots\ldots\ldots (a)$$

To find f_s at any other point, put 2_x in place of L_1. This equation gives larger unit stresses in the side belt rods of the head than Equation (42), but will itself need to be reduced somewhat in case the belt under consideration has an unusual number of laps in the head. However, it leaves the unit stresses determined by Equation (34) unchanged. The reason for these larger stresses is the longer cantilever span due to the location of points of inflection.

As the law of the distribution of stresses in the rods of a diagonal belt differs from that in a side belt, where all the rods have an unequal stress, the writer will obtain an analogous value of the stress in the rods of a diagonal belt. Starting with Equation (39), which refers to the part of the panel forming the column head, and transforming to axes, x' and y', along the diagonals, as was done in Equation (47), an expression will be obtained which is identical with Equation (47), except that g will be replaced by $a + b$.* Hence, in place of Equation (49), we have for this area:

$$\pm\, e' = id\, \frac{\delta^2 z}{\delta\, x'^2} = \frac{(1 - K^2)\, q\, (a + b)}{24\, E j\, d_3\, \Sigma A} \left[a^2 + b^2 - 3\, (x'^2 + y'^2)\right]$$

* There is a misprint in the last term of Equation (47) in the writer's book, and $x'^2 + y'^2$, should be substituted for $x'^2 y'^2$,

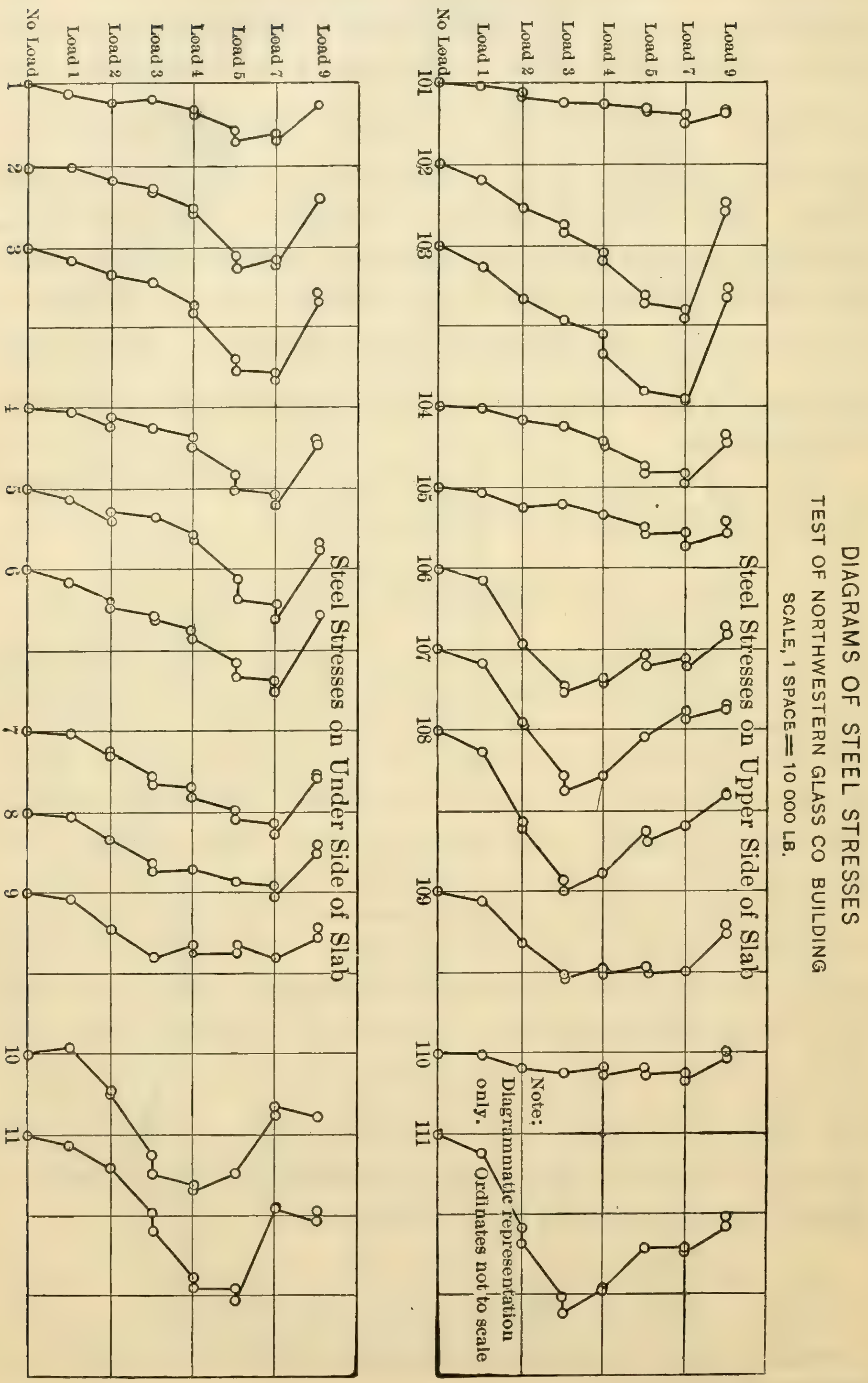

DIAGRAMS OF STEEL STRESSES
TEST OF NORTHWESTERN GLASS CO BUILDING
SCALE, 1 SPACE = 10 000 LB.
Steel Stresses on Upper Side of Slab
Steel Stresses on Under Side of Slab
Note:
Diagrammatic representation only. Ordinates not to scale
No Load
Load 1
Load 2
Load 3
Load 4
Load 5
Load 7
Load 9
101
102
103
104
105
106
107
108
109
110
111
1
2
3
4
5
6
7
8
9
10
11

FIG. 12.

In this, by Equation (41), $j \Sigma A = 6 A_1$. Introduce the effect of the size of the cap in the same manner as in Equation (*a*), derived for side belts, and apply it to find the stress at the edge of the belt opposite the column center, where:

$$4 x'^2 = L^2 = L_1^2 + L_2^2 = 4 (a^2 + b^2), 4 y'^2 = g^2$$

$$4 (x'^2 + y'^2) = L^2 + g^2 = L'^2, \text{ and,}$$

$$C_1 = \frac{1}{4} \left(\frac{L_1}{L_2} + 1\right) \left(1 + \frac{L_2^2}{L_1^2}\right) = 1, \text{ nearly, then,}$$

$$f_s = Ee' = \frac{C_1 W L_1}{400 d_3 A_1} \left(\frac{3 L_2^2}{B'^2} - 1\right) \dots\dots\dots\dots\dots\dots (b)$$

where B' is the clear span along the diagonal between the caps. This equation gives the stress in the rod at the edge of the diagonal belt opposite the column center, and this is assumed also to give the value of the unit stress in the middle rod of the belt just outside the circular cap. To find the stress at any other point, x' y', of the diagonal belt in the head, substitute $4 \left(x'^2 + y'^2\right)$ for L'^2.

It will be noticed that in the central area of the panel, where moments are positive, the stress in the diagonal rods is greatest in the middle rod, but, in the column heads, where moments are negative, the stress is greatest in the rods at the edge of the belt.

The value of the greatest tensile stress at the edge of a direct or diagonal belt, found from Equations (*a*) or (*b*), however, is computed on the assumption that the steel in the slab is practically at the same depth as the rods nearer the center of the same belt. Such, however, is not ordinarily the case. The edges of the belts near the cap lie at a somewhat lower level than the remainder of the belt, and, being nearer the neutral axis, the rods at the edges, for that reason, will not be subject to as large a stress as is computed from Equations (*a*) or (*b*), and may not have as large stresses as rods somewhat nearer the center.

In the rods of the diagonal belts, points of inflection and points of equal stress lie on circles with centers at the center of the panel, so that the points of inflection of the side belts which lie on lines perpendicular to the sides do not coincide with the points of inflection of the diagonal rods.

In this test no observations were made on the rods of the side belts by which any comparison can be made with Equation (*a*), but several points on the diagonals near the edges of the caps were ob-

served, which may be estimated by Equation (b), though no observations were made on the rods near the edges of the diagonal belts.

Under Load 3 it will be assumed that each of the four panels was loaded with what would be equivalent to a uniformly distributed total

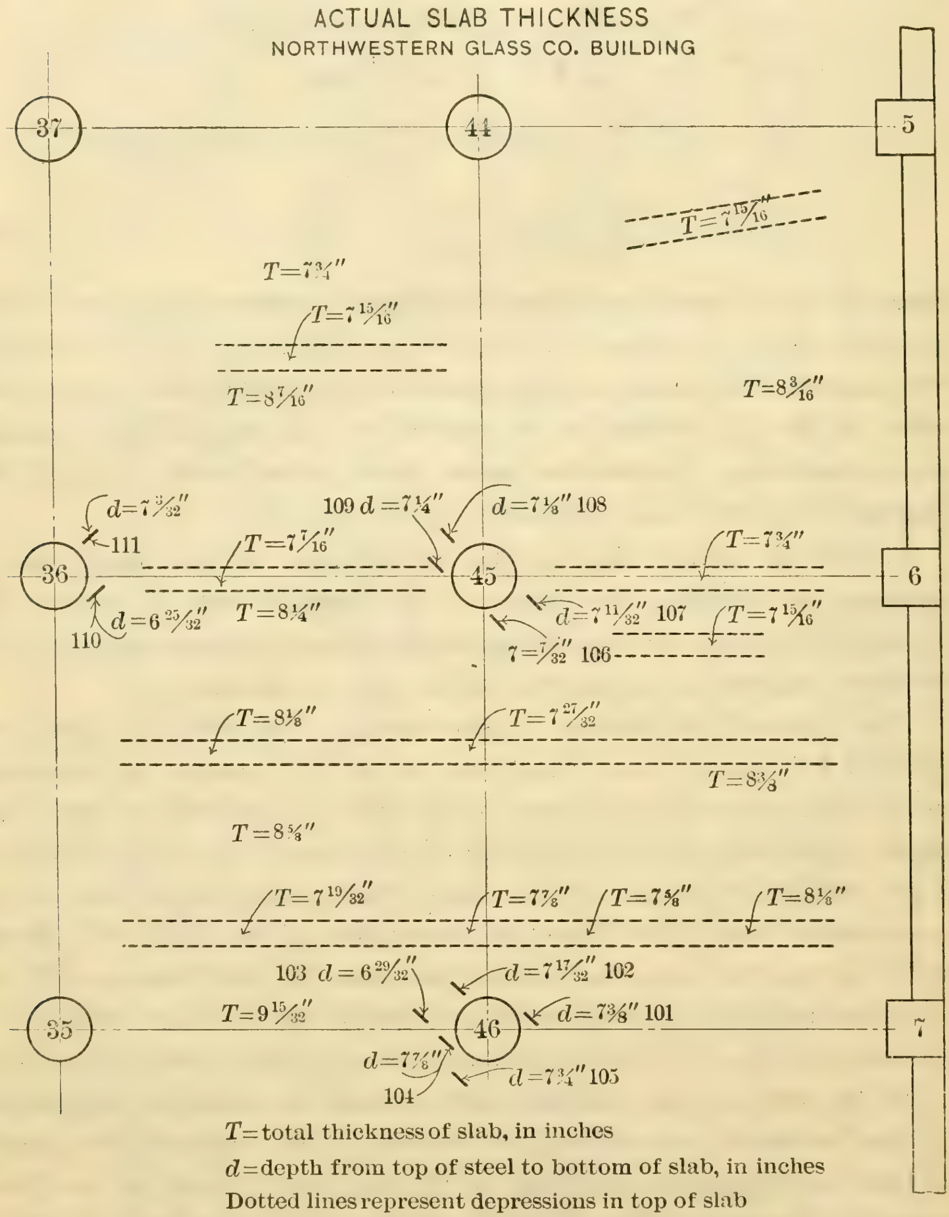


FIG. 13.

load of $W = 106\,250$ lb. By Equation (b) the limiting value of the unit stress in the outer rod is

$$f_s = \frac{106\,250 \times 204}{400 \times 6.5 \times 1.6567} \left(\frac{3 \times 86\,049}{60\,025} - 1 \right) = 17\,000 \text{ lb. per sq. in.}$$

This is to be compared with the stresses observed on Fig. 3, most especially at 107 and 108, around the central cap of the loaded area, which were, respectively, 17 500 and 20 000 lb. Incidentally, comparisons should be made with observations at 102 and 111, at the caps on the edge of the loaded area.

Good agreement of the computed with the observed results is hardly to be expected in this case, because, just as the concrete was about to be poured, it was discovered that the radial elbow rods, by some mistake, had not been bent at quite the correct angle to correspond with the thickness of the slab, and it was necessary to pry the ends of the rods upward forcibly and hold them in this position by blocks under their extremities, thus putting them under considerable initial

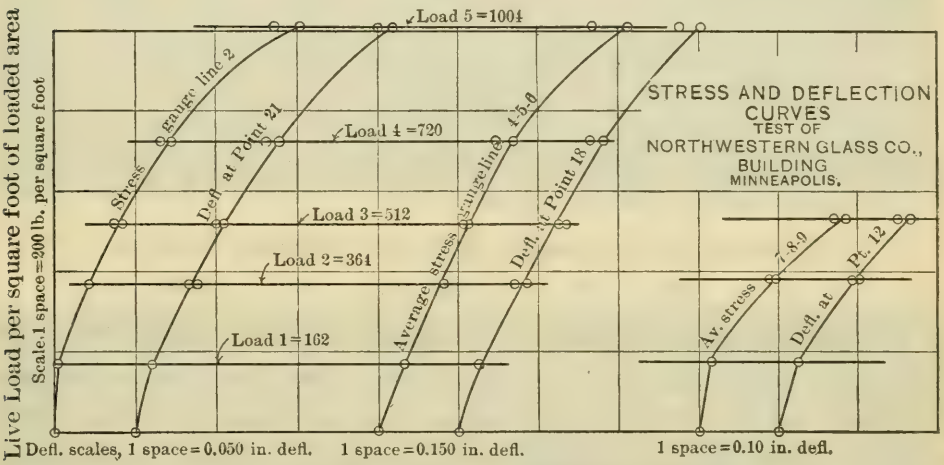


FIG. 14.

bending stress. The relief of this bending by the test load would influence the observations to some extent, and tend to make the observed values of f_s in the slab rods larger than those computed. At 111, Column 36, on the edge of the loaded area, the abnormal value of 22 400 lb. was reached under Load 3. Load 5 shows no such large increase over Load 3 of observed stress in the slab rods of the column heads in Panel D as was to be expected by practically doubling the loading. This fact is apparently inexplicable.

By reason of the disarrangement of the stresses due to initial bending, as mentioned, it seems to be impossible to trace the stresses in the slab rods in greater detail in this test than has been done already, where a satisfactory concordant maximum stress in the slab rods has

been reached. Consequently, a comparison of the computed with the observed deflections will now be considered. By Equation (71)

$$D_2 = \frac{W L_2 \, L_1^2}{4.4 \times 10^{10} \, A \, d_2^2}$$

in which $d_2 = 8 - 0.5 - 0.5625 = 6.9375$ in.

Hence $$D_2 = \frac{200\,000 \times 16 \times (17)^2 \times 1\,728}{4.4 \times 10^{10} \times 1.6567 \, (6.9375)^2} = 0.4555 \text{ in.}$$

This agrees precisely with the value observed at 9 A. M. on May 18th under Point 18 at the center of Panel D. Practically the same deflection was found at the adjacent Point 17.

By Equations (61), (54), and (58), the deflection at the middle of the side belt is

$$D_1 = \frac{W \, L_1^3}{10^{10} \, A} \left[\frac{1}{10.7 \, d_1^2} + \frac{\frac{L_1}{L_2} + 1}{60 \, d_3^2} \right]$$

in which $d_1 = 7.31$ in. and $d_3 = 6.5$ in.

The values of the deflections are:

At Point 12: computed, 0.27 in., observed, 0.22 in.
" " 20 " 0.25 in., " 0.20 in.

The magnitude of the observed deflections in the side belts is less than those computed, for the same reason that the stresses are less, as before stated, although it is clear that the observed values would possibly reach those computed in case all the panels were loaded equally. This last remark is confirmed by considering the deflection under Load 3, which was nearly the same on all four panels, and equivalent to 106 250 lb., uniformly distributed, as found previously. The computed deflections would then be somewhat more than one-half of those already computed, and may be compared with the observed values, as follows:

At Point 12: computed, 0.15 in., observed, 0.16 in.
" " 20 " 0.13 in., " 0.13 in.

From which it may be concluded that the computed results would not be less than those observed if all the panels were equally loaded.

It seems clear that the continued increase of the deflection at Point 18 at the center of Panel D under Load 7 was influenced somewhat by the removal of the loading in Panel B, and that changes in the de-

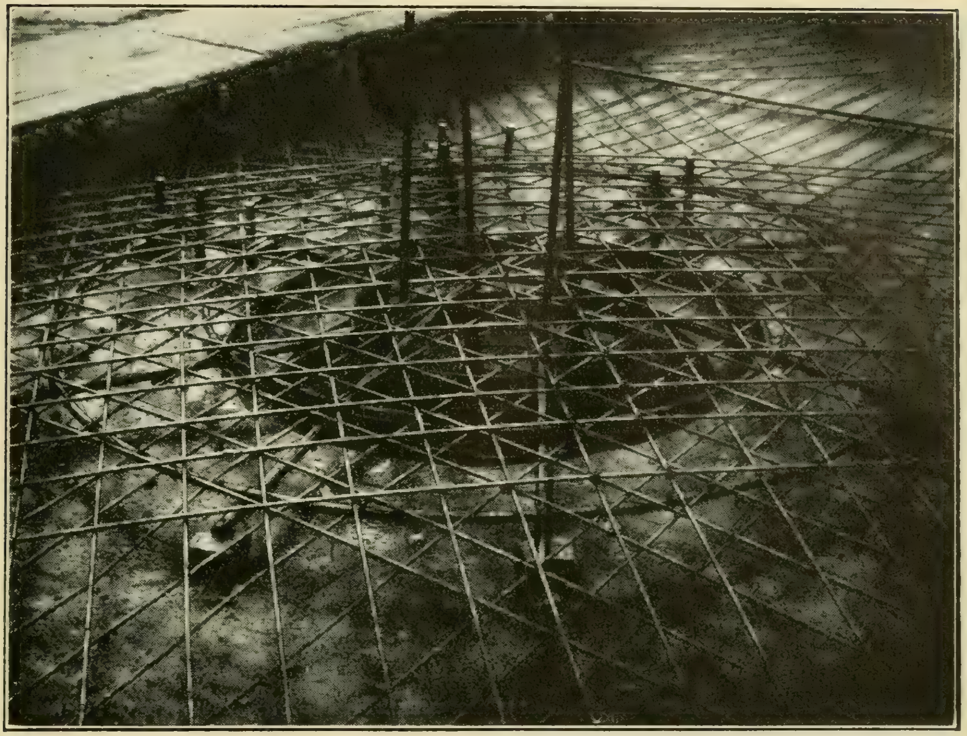


FIG. 15.—SLAB REINFORCEMENT, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

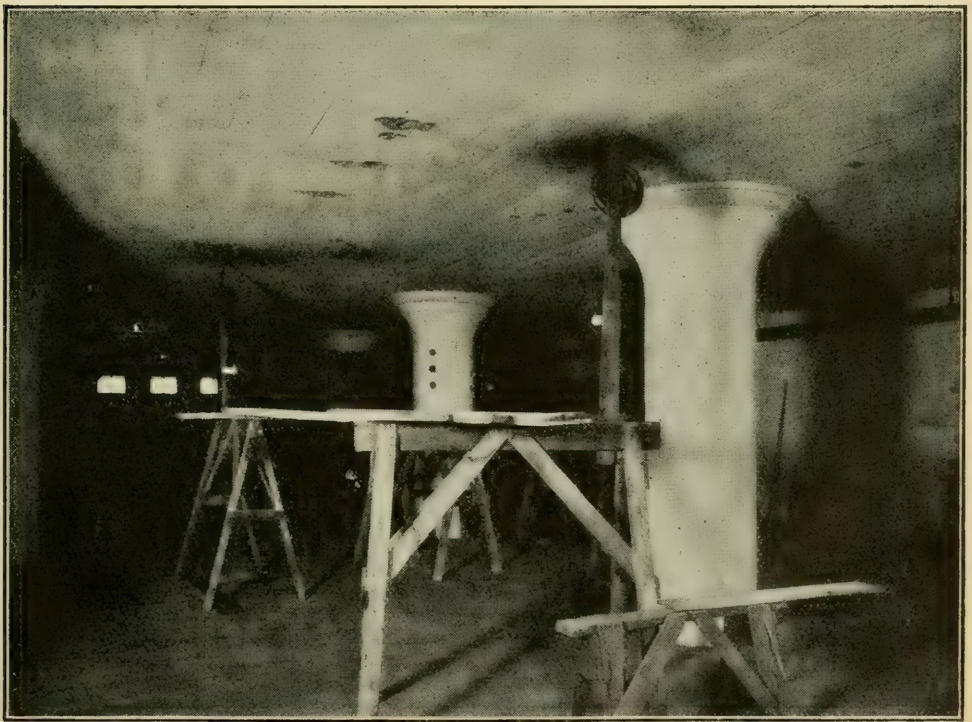


FIG. 16.—UNDER SIDE OF TEST PANELS, NORTHWESTERN GLASS COMPANY BUILDING, MINNEAPOLIS.

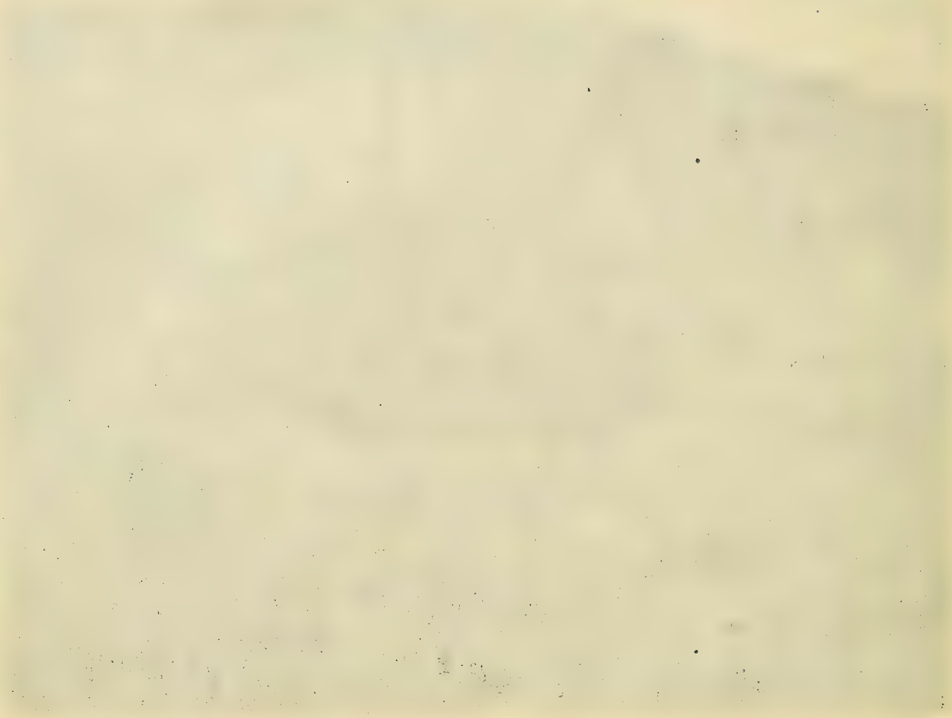


FIG. 17.—UNIFORM LOAD OF 400 LB. PER SQ. FT. OVER FOUR PANELS. NORTHWESTERN GLASS COMPANY BUILDING.

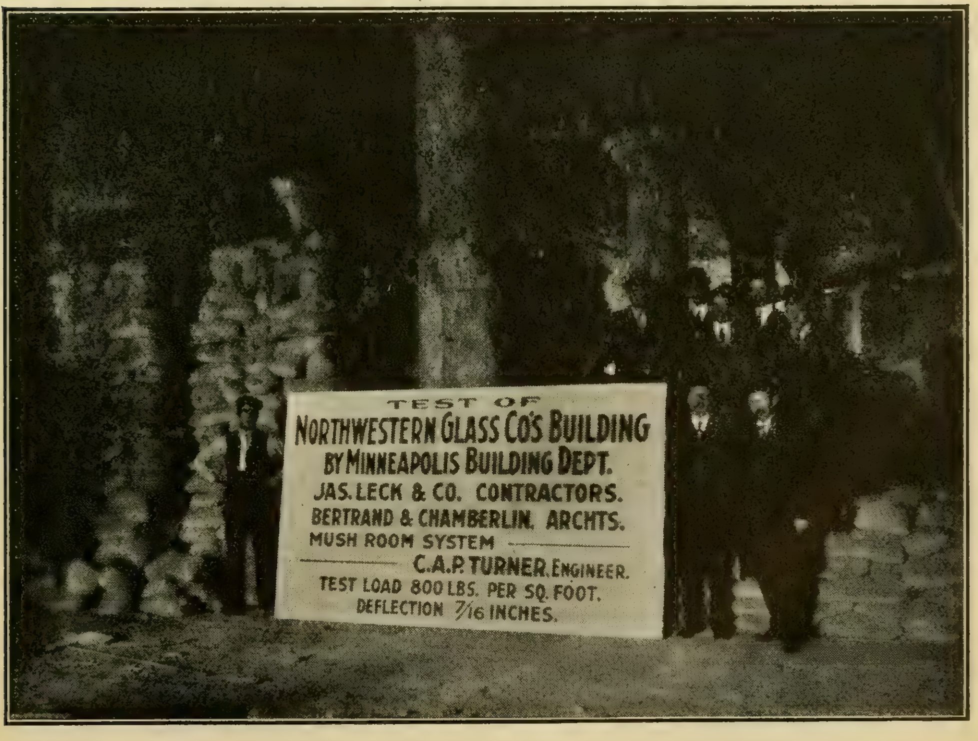


FIG. 18.—TEST OF NORTHWESTERN GLASS COMPANY'S BUILDING BY MINNEAPOLIS BUILDING DEPARTMENT.

flections of adjacent panels may be looked for along the prolongation of diagonals rather than across the sides of the panels. This is perhaps the first test of a mushroom slab in which it has been possible to discover interaction of panels. Possibly, however, this may be due to the initial stresses in the radial steel previously mentioned.

Fig. 7 is of interest as showing the dissymmetry in the right and left half spans of the diagonals in Panels A and D. In A the supporting action of the wall along the entire length of the side evidently prevented deflection to some extent in the half span nearest to it, so that Point 9 was deflected less than the corresponding point in the other half span. In Panel D Point 18 was deflected less than the corresponding Point 16, because all the slab rods are lapped over Column 35, and none over 45, as may be seen on Fig. 1.

Owing to the irregularities of loading and variations of stiffness due to the inclusion of the wall panels, as well as accidental variations of stiffness of construction due to initial bending stresses, and also the inequalities of laps, it is too much to expect formulas derived for entire uniformity of loading and equality of panels to apply more closely than these comparisons show.

The Deere and Webber Company Building.

The test of the Deere and Webber Building, in Minneapolis, Minn., was made on October 30th to November 5th, inclusive, 1910, by Mr. A. R. Lord.* The panel dimensions are 19 ft. 1 in. by 18 ft. 8 in., the slab is $9\frac{3}{16}$ in. thick, and the concrete was only 40 days old on October 30th. All the slab rods are $\frac{7}{16}$ in. round, with twelve rods in each side belt and fourteen in each diagonal. The head steel consists of the rectangular diamond frame with four rods extending entirely across it, which was at that time the typical form used by the Leonard Construction Company in such buildings. The column caps are 54 in. in diameter and the slab rods of all belts are 7.25 in. from center to center, so that the side belts may be taken to have a width of 80 in. and the diagonals nearly 95 in. The mean size of the cap is 0.24 L.

Various loads, uniformly distributed, were applied simultaneously to eight panels, which, with the omission of one corner panel, filled

*Reported by Mr. Lord in a paper printed in *Proceedings*, Nat. Assoc. of Cement Users, Philadelphia, Vol. VII, 1911.

out a square of nine panels. This gave opportunity to observe stresses in the slab rods in one panel at the center which was surrounded on all sides by equally loaded panels.

This discussion will refer mostly to the results observed when the greatest load was on all the eight panels at once. This load had a mean intensity of 350 lb. per sq. ft. The area of a panel is $356\frac{2}{9}$ sq. ft., and the total load $W = 350 \times 356\frac{2}{9} = 124\,678$ lb. The depth of the embedment of the various rods observed is given in the report, but, as the rods of a belt act together, it is not certain that the embedment of individual rods should be used in the formulas. It has been assumed that the fair mean values are

$$d_1 = 8.5 \text{ in.}, d_2 = 8 \text{ in., and } d_3 = 7.6 \text{ in.}$$

Applying Equation (34) to compute the unit stress in the slab rods on the short side of the panel at Point 14, of Fig. 3, in Mr. Lord's paper, which is the point where the greatest stress was found in any slab rod of a side belt between two loaded panels,

$$f_s = \frac{124\,678 \times 224}{175 \times 8.5 \times 12 \times 0.15} = 10\,000 \text{ lb. per sq. in.}$$

The observed stress at this point was 10 400 lb. The unit stress observed at Points 39, 40, and 14*a*,* on the short side belts, were somewhat less, but this is a satisfactory determination of the greatest stress at the middle of a short side belt.

Applying Equation (34) to compute the unit stress in the long side belt at Points 108 and 109,

$$f_s = \frac{124\,678 \times 229}{175 \times 8.5 \times 12 \times 0.15} = 10\,220 \text{ lb. per sq. in.}$$

The mean observed result was 6 600 lb. It is probable that this result, observed between two loaded panels bounded by panels not loaded, would be less than on the same belt in the two loaded panels beside it, on the principle that if only three successive spans of a continuous beam of many spans be loaded, the middle one will show less stresses and deflections than the end spans.

This, together with the fact that the inflection lines were much nearer to the center of the panel, was probably the reason that the observed result was so small. Apparently, however, there is no reason

* These refer to Fig. 3 of Mr. Lord's paper.

for the greatest unit stress being less on the long side of the panel than on the short side, when the belts are equal and the panels are so nearly square. In this particular the results of the test seem to be inexplicable unless the two bulkheads which extend nearly across the slab parallel to the long sides of the panels, one of which crosses the short side at 14, have this effect.

A similar discrepancy was found between the observed stresses on the long and short sides of the panel in other tests. It is certain that provision should be made on this long side for stresses at least as great as those on the short side.

Applying Equation (52) to the computation of the unit stress in the rods of the diagonal belt at the center of the panel at Points 3, 7, 110, and 12*a*, gives

$$f_s = \frac{124\,678 \times 229}{256 \times 0.89 \times 14 \times 0.15 \times 8} = 7\,440 \text{ lb.}$$

There seems to be some discrepancy between the table of numerical values and the plotted maximum stress at Point 12*a*. In any case, 12*a* shows abnormally large readings compared with the others, but the computed value just found is greater than three of the observed values and greater than the mean of all four.

The diameter of the cap being $0.24L$, the clear span between the edges of the caps on the side belt is $0.76L$, and the least computed distance of the points of inflection along the middle of the side belt is this divided by $\sqrt{3}$, or $0.577 \times 0.76L = 0.44L$; and, consequently, the distance between the points of inflection across the column center is $0.56L$, which, as stated by Mr. Lord in a second paper,* is precisely the observed position of the points of inflection in the side belt of this slab.

Applying Equation (*a*) to the computation of the unit stress at Point 205, Fig. 3, of Mr. Lord's paper, on the edge of the long side belt, gives

$$f_s = \frac{124\,678 \times 229 \times 453}{800 \times 7.6 \times 13 \times 0.15 \times 224}\left[\frac{3 \times 52\,441}{30\,625} - 1\right] = 20\,150 \text{ lb.}$$

per sq. in.

The value observed at 205 was 20 000 lb., and slightly less at the symmetrical position, 208, also situated at the edge of a short side

* Also published in *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, 1911.

belt. The corresponding unit stress in a rod tangential to the edge of the cap at 206 was less, for the reason previously stated, being only 14 400 lb. at 206, and 12 300 lb. at 209.

Applying Equation (*b*) to the computation of the limiting unit stress in the rods at the edge of the diagonal belt opposite the column center gives

$$f_s = \frac{124\ 678 \times 229}{400 \times 7.6 \times 13 \times 0.15}\left[\frac{3 \times 111\ 642}{70\ 756} - 1\right] = 22\ 440 \text{ lb. per sq. in.}$$

All the observed stresses were somewhat less than 16 800 lb., both at 202, the outside rod, and at 204, the rod tangent to the cap, and 18 300, at 203, the rod lying between them. The outside rod, for the reason already stated, did not show quite the computed stress, but the observed stress in the center rod of the belt, where it crossed the edge of the cap at 207, was 23 400 lb., when all the panels were loaded, and 24 200 lb. when half the load had been removed from the outer panels.

Applying Equation (71) to the computation of the deflections at the center of the panel gives

$$D_2 = \frac{124\ 678 \times 224 \times 229 \times 229}{4.4 \times 10^{10} \times 64 \times 14 \times 0.15} = 0.248 \text{ in.}$$

Point 37, Fig. 3,* was the only center point observed in a loaded panel wholly surrounded by loaded panels. The mean value of seven readings was 0.224. When half the load was removed from the outside panels the deflection increased to 0.246 in. The mean readings at the centers of adjacent panels were: Point 4, 0.291 in.; Point 8, 0.306 in.; Point 12, 0.271 in.; all these being somewhat larger because adjacent panels were not loaded.

Applying Equations (54), (58), and (61) to the computation of the deflections at the middle points of the side belts gives $D_1 = 0.1555$ for the long side, and $D_1 = 0.1456$ for the short side. The only reading at the middle point of a long side belt not continuous with or contiguous to an outside panel was at Point 17 where the mean of seven readings was 0.1387. The mean reading at Point 5, at the middle of the continuation of the same rod on the other side of Column 41, between two loaded panels, was 0.229. These points were situated similarly to the middle points of three successive equally loaded spans

* Of Mr. Lord's paper.

of a continuous beam, Point 17 being at the middle point of the middle span, and Point 5 at the middle of one of the end spans; the first deflection was somewhat less, and the other somewhat greater than the deflection when many spans were equally loaded. Point 41 was situated so that its mean deflection should fall between that at 17 and that at 5, which is the fact.

Point 38 was on a short side, on a location in the middle span of three spans like that of 17 on the long side, and the mean reading was 0.127; Point 15 was on a short side, as 5 was on a long side, in an end span of three spans, and the mean reading was 0.163. The computed deflection falls between these, as it should. The deflection at 42 should fall between the readings for Points 38 and 15, which is the fact. Point 11 was in practically the same relative position as 15, and had equal deflections. Other deflections at the middle of outside belts had about half the preceding deflections, influenced more or less by their continuities.

The interaction of successive spans across the column heads, by reason of continuity of spans, is evidently of considerable amount in this floor. This is doubtless due to lack of stiffness at the junction between columns and slab, which in this design is almost entirely without stiffening rods, such as the elbow rods in the mushroom system. The very striking agreement of these computations with the observations in this test show that in all essential particulars the action of the reinforcement is in principle identical with that for which the equations here applied were originally developed.

The Larkin Building.

A very extensive and painstaking test of the Larkin Building, in Chicago, was made by Mr. A. R. Lord.* The design of the slab reinforcement in this building was of the general four-way type used in the mushroom system, but the column heads omitted much of the steel ordinarily used for stiffening the head, and, in place of it, resistance to bending and shear was supplied by a depressed head or "drop" of concrete, 8 ft. square and 6.75 in. thick, placed at the top of each column, between the cap and the bottom of the slab. This was made integral with the slab and formed part of it, just above the cap. The panels are 20 ft. by 24 ft. 2 in., or $L_1 = 290$ in., and

* The results were presented by Mr. Lord in a paper at the Ninth General Convention of Cement Users. Extracts from this appear in the *Cement Era*, January, 1913, p. 52.

$L_2 = 240$ in. The thickness of the slab proper is 9 in., except at the drop, where it is 15.75 in. The diameter of the cap is 60 in. The width of the side belts is from 90 to 93 in., and of the diagonal belts from 105 to 108 in. All the slab rods are ½-in. round, 13 in each short side belt, 22 in each long side belt, and 21 in each diagonal belt.

The floor was designed for a dead load of about 120 lb. per sq. ft., and a live load of from 225 to 250 lb., with a maximum test load of twice the sum of these, or actually 738 lb. per sq. ft. As the dead weight of the slab was acting at all times, the stresses tabulated in the report are those due to the observed elongations plus corrections due to the weight of the slab itself, which is included in the 738 lb. The deflections given, however, are those actually observed under the loads applied, or a maximum test load for deflections of about twice 250 lb., plus 120 lb., or a total actual maximum load of 618 lb. per sq. ft. Hence the total maximum load producing stress was $W = 738 \times 20 \times 24\frac{1}{6} = 356\,700$ lb. The load was applied, and the readings of stresses and deflections were made at several stages. At Stage 2, 130 lb. per sq. ft. was applied over a test area of five panels, three of them wall panels, and two adjacent to them, not wall panels. In Stage 3 the load on this area was increased to 250 lb. In Stage 4, only three panels were loaded, two of them wall panels and one adjacent interior panel, with 415 lb. per sq. ft. Finally, in Stage 5, the entire load of 618 lb. per sq. ft. was placed on two panels only, one wall panel, and an adjacent interior panel, and these were situated so that their short sides were contiguous.

Apply Equation (34) to the computation of the unit stress in the rods of this short side belt, in which, ordinarily, it would be found that $d = 8.25$ in., but among the test data this is stated to have a value of 8 in. in this case. Hence

$$f_s = \frac{356\,700 \times 240}{175 \times 8 \times 13 \times 0.196} = 24\,000 \text{ lb. per sq. in.}$$

The observed stress was 24 200 lb., which would be decreased somewhat, as noted previously, if adjacent panels were also equally loaded, as contemplated in Equation (34). Applying Equation (34) to the long side belt,

$$f_s = \frac{356\,700 \times 290}{175 \times 8 \times 22 \times 0.196} = 17\,000 \text{ lb. per sq. in.}$$

The observed stresses in the rods of a belt lying between a loaded panel and one not loaded is much less than between two loaded panels. The observed value in this location is given at 10 200 lb., but it is not stated whether this is a mean value or the greatest value for any rod of the belt, or the stress in the middle rod. From the figured location of the test lines, it is probably the latter. The computed results should be regarded as in most satisfactory agreement with the observations.

Applying Equation (52) to the computation of the unit stress in the middle rod of the diagonal belt at the center of the panel:

$$f_s = \frac{356\,700 \times 290}{256 \times 0.9 \times 7.75 \times 21 \times 0.196} = 14\,070 \text{ lb. per sq. in.}$$

in which 7.75 in. is taken as the mean distance of the center of the steel from the upper surface at the center of the panel. The given observed value was 12 900 lb., which differs from the computed by less than 10 per cent. It is not stated whether the observation was in the wall panel or in the interior panel, or whether it was an average or in the middle rod of the belt, but it was probably the latter. If so, the stress in the middle rod should be greatest. This result may be regarded as satisfactory, and very near what would be observed in the middle rod if the adjacent panels were equally loaded.

Before computing the stresses in the head steel at the edge of the cap and edge of the drop, it should be noted that all the stresses given in the general summary of stresses for the maximum load of 738 lb. are from 2½ to 3 times as great as for the design load, which is half as great. This, among other things, signifies that the value of the modulus of elasticity for the concrete was much less under the test load than was its initial value. Under the test load of 738 lb., there is no one of the columns entirely surrounded by loaded panels. The consequence is that the observed stresses in all the belts of the head will be much less than those computed, and if an attempt were made to compute the stresses due to the design load which was spread over all the panels around the column, where most of the readings on the head belts were evidently made, the observed stresses would have to be multiplied by a coefficient of 1.5 or more to make them comparable with the results at the test load, 738 lb.

Now write Equation (*a*) in the general form

$$f_s = \frac{W L_1 (L_1 + L_2)}{800\, d_3\, A_1\, L_2}\left[\frac{3\,(2\,x)^2}{B_1^2} - 1\right] \ldots\ldots\ldots\ldots (a)$$

in order to compute the unit stress in the rods of the long side belt at the edge of the cap, where $2x = B_1 = 290 - 60 = 230$ in., and $d_3 = 14$ in. The drop is so thick as to obviate mostly the increase of stress in the center rod where it crosses the edge of the cap, and therefore no account is taken of such increase. This unit stress is found to be $f_s = 9\,460$ lb. per sq. in. The observed stress along this side at the edge of the load, of course, was considerably smaller, and is given as 7 000 lb.

Making a similar computation at the edge of the drop on this belt, where $B_1 = 230$ in., $2x = 290 - 96 = 194$ in., and $d_3 = 7$ in., it is found that $f_s = 14\,500$ lb. per sq. in. Along this side belt at the edge of the load the observed result was only 10 400. The diagram of gauge lines seems to show that these are readings observed along the middle rod of the belt. In that case the outside rod at the edge of the belt under a load over many panels would have shown greater unit stresses than those computed, or, in the writer's opinion, at least as large.

Applying Equation (*a*) to the computation of the unit stresses in the short side belt at the edge of the cap, by exchanging the subscripts, 1 and 2, and placing $2x = B_2 = 240 - 60 = 180$ in., $d_3 = 14$ in., and $A_2 = 13 \times 0.196$, gives $f_s = 10\,950$ lb. per sq. in. This belt crosses the cap from the unloaded into the loaded area which covers the two panels on each side of it. The tipping of the head relieves to some extent the stresses which would exist were all panels loaded, and the reading given is 7 300 lb.

Again, compute the unit stress at the edge of the drop in this short side belt by inserting in the equation just used $B_2 = 180$ in., $2x = 240 - 96 = 144$ in., $d_3 = 7$ in., and $f_s = 15\,300$ lb. per sq. in. At this point there is something abnormal and inexplicable in the observed results which, apparently, should be larger than those observed at the edge of the cap where the concrete is so thick. This contradictory reading is only 5 800 lb., and may be due to some bend in the rod, or other peculiarity, because the same thing appears at the smaller loads.

Writing Equation (b) in the general form

$$f_s = \frac{C_1 \, W \, L_1}{400 \, d_3 \, A}\left[\frac{3\,(2\,x')^2}{B'^2} - 1\right] \qquad (b)$$

and computing the unit stress in the middle rod at the edge of the cap on the diagonal of 21 rods by increasing B' between the corners of the assumed square caps and placing

$$L = \sqrt{290^2 - 240^2} = 320 \text{ in.}, \; B' = 320 - 85 = 235 \text{ in.},$$

$$2x' = 320 - 60 = 260 \text{ in.}, \; f_s \text{ is found to be as follows:}$$

$f_s = 12\,480$ lb. per sq. in. The necessarily smaller observed value was 10 800 lb.

Apply the same equation to compute the unit stress in the middle rod of the diagonal belt at the edge of the drop, assumed to be 96 in. in diameter because the projecting corner should be disregarded, and we have $2x' = 320 - 96$ in., $B' = 235$ in., $d_3 = 7$ in. Hence $f_s = 16\,160$ lb. per sq. in. The smaller observed value was 14 200 lb.

Applying Equation (70) to compute the deflection at the center of the panel gives

$$D_2 = \frac{W \, L_2 \, L_1^2}{10^{10} \, A_1}\left[\frac{1}{6.56 \, d_2^2} + \frac{1}{12.5 \, d_3^2}\right]$$

in which the total load producing the observed deflection was 618 lb. per sq. ft., or a total load of $W = 296\,850$ lb.

$$D_2 = \frac{296\,850 \times 240 \times 290^2}{10^{10} \times 22 \times 0.196}\left[\frac{1}{6.56 \times 7.75^2} + \frac{1}{12.5 \times 14^2}\right]$$

or $D_2 = 0.41$ in. The observed deflections at Points 10 and 16 were 0.40 in., an extraordinary agreement, under the circumstances, of reduction of steel in the head and displacement of inflection lines which produce effects that tend to balance each other.

Deflections at the middle of the long side may be computed by Equations (91), (54), and (58), as follows:

$$D_1 = \frac{296\,850 \times 290^3}{10^{10} \times 22 \times 0.196} \; \frac{1}{10.7 \times 8^2} \; \frac{2.2}{60 \times 14^2} = 0.276 \text{ in.}$$

The observed deflections were 0.21 in. at Point 15 and 0.26 in. at Point 17, on the edges of the loaded area, which values would be somewhat increased if the adjacent panels were also equally loaded. A similar computation at the middle of a short side belt gives results still more in excess of the value observed at Point 13, Fig. 3, of Mr. Lord's paper,

by reason of the shortness of the concave section between the points of inflection. The surprising thing is, not that such discrepancies exist between computed and observed results, but that equations derived for a flat slab differing so widely and in so many particulars from this one should nevertheless be applicable at all.

The writer's equations were derived for the case of side belts with total cross-sections proportional to their spans and with diagonals with intermediate cross-sections which, according to his theory, is the correct proportion. The Larkin slab, however, has the cross-sections of its side belts proportional to the cubes of their spans, which is a highly uneconomical arrangement of steel, as shown by the unit stresses observed in the test. The fact that these equations apply so closely shows that the use of the drop in stiffening the head is practically identical in action with the steel frame of the column head which it replaces.

In view of the unqualified assertion made in the extract from Mr. Lord's paper in the *Cement Era,* respecting the Larkin test, that "the deflection readings, compared with those of other tests, show that the resulting structure is stiffer and stronger than the straight flat slab as ordinarily designed, with anywhere the same amount of materials", it becomes of interest to submit such a straight flat slab to analysis for purposes of comparison, as the writer's formulas are applicable to such a slab with considerable accuracy.

It is to be noticed that deflection readings do not of themselves permit any conclusions whatever to be drawn as to the strength of a structure, but have reference to stiffness only, so that that half of the assertion referring to strength is, on the face of it, unwarranted and entirely unsupported by these readings, and, in fact, will be found to be incorrect, as the writer will proceed to show. As to deflections, it may be that a perfectly flat slab might have a deflection of perhaps not more than 10% in excess of this slab, with its bulky and unsightly drop at the head of each column. If that should possibly be the case, it is a trivial matter, and of no consequence compared with the very serious loss of strength involved in this construction.

The volume of the drop, which is 8 ft. square and 6.75 in. thick, is 62 200 cu. in. In case this quantity of concrete were used to increase the mean thickness of the slab, it would make a flat slab approximately 0.9 in. thicker than at present. Such a flat slab 9.9 in. thick through-

out, with belts of 0.5-in. round slab rods, having 19 rods on each short side, 19 in each diagonal belt, and 22 on each long side, would have slightly less slab steel, and the same quantity of concrete in the slab as in the Larkin Building, but the maximum stresses in the steel would be far less.

The unit stress in the slab rods at the middle of the long side belt of this straight flat slab is found by Equation (34) to be

$$f_s = \frac{W L_1}{175\, d_1\, A_1} = \frac{356\,700 \times 290}{175 \times 9 \times 22 \times 0.196} = 15\,230 \text{ lb. per sq. in.}$$

Similarly, the unit stress at the middle of the slab rods in the short side belt is

$$f_s = \frac{W L_2}{175\, d_1\, A_2} = \frac{356\,700 \times 240}{175 \times 9 \times 19 \times 0.196} = 14\,600 \text{ lb. per sq. in.}$$

By Equation (52) the unit stress in the diagonals at the center of the panel is

$$f_s = \frac{W\, L_1}{256\, j\, d_2\, A_2} = \frac{356\,700 \times 290}{256 \times 0.89 \times 8.56 \times 19 \times 0.196} = 14\,400 \text{ lb. per sq. in.}$$

It would require a load of more than 1 200 lb. per sq. ft. on such a mushroom slab to produce a maximum unit stress on the slab rods as great as was caused in the Larkin slab by the test load of 738 lb. per sq. ft. The difference is so great as to nullify entirely any such unwarranted claims as those quoted in the extract from the *Cement Era.*

It thus appears that a mushroom slab more than 65% stronger than the slab in the Larkin Building can be constructed at the comparatively slight additional expense of part of the steel required for the radial and ring rods of the mushroom head. How much more steel might be required for the mushroom is unknown, because no data are available as to the quantity of steel used in the top rods which were embedded in the top of the slab over the column heads, and in the frame supporting the slab rods in the column heads in the Larkin slab. In Mr. Lord's paper a photograph of the steel in place for pouring the concrete shows a rectangular frame support of this kind. Any claim for superiority of the Larkin structure with reference to strength and economy of materials over a properly designed "straight" flat slab is, in the writer's opinion, wholly without foundation.

The Donnelly Building.

The Donnelly Building, in Chicago, Ill., was designed and tested by Mr. A. R. Lord. Certain data of this test, as yet unpublished, have been put at the writer's disposal by C. A. P. Turner, M. Am. Soc. C. E. The panels in the slabs are 24 ft. 10 in. by 24 ft. 4 in. and 11 in. thick, with a drop of concrete at the head of each column, 9 ft. square and 9 in. thick, integral with the slab. The total thickness of the slab at the edge of the cap is 20 in.; the effective thickness from the center of the reinforcement in the head to the lower surface of the slab is 18 in.

Design load, 300 lb. per sq. ft.;
Weight of slab, 150 lb. per sq. ft.;
Test load, 750 lb. per sq. ft.;
Area of panel, 604.3 sq. ft.;
$L_1 = 298$ in.;
$L_2 = 292$ in.

All the reinforcement was of ½-in. round rods. Each long side belt had 26 rods, and each short side belt and diagonal belt, 25 rods. The computed maximum unit stress at the center of the short side belt, by Equation (34), is

$$f_s = \frac{292\ W}{175 \times 25 \times 0.196 \times 10.25}.$$

For a working load of 300 + 150 = 450 lb. per sq. ft., $f_s = 9\,000$ lb. For the total test load of double this quantity, that is, 750 lb. per sq. ft., in addition to the weight of the slab itself, $f_s = 18\,000$ lb. when all the panels are equally loaded.

The deflection at the center of the panel may be computed for the test load of 750 lb. per sq. ft. by Equation (70) as follows, by assuming $\Sigma A = 6A_1$ as the reinforcement in the area over the head of the columns, this being 80% of that assumed in the standard mushroom design:

$$D_2 = \frac{604.3 \times 750 \times 292 \times 298^2}{10^{10} \times 25 \times 0.196}\left[\frac{1}{6.56 \times 9.75^2} + \frac{1}{10 \times 18^2}\right] = 0.384 \text{ in.}$$

The observed deflection was barely 0.3 in., which shows that the lines of inflection in this design were somewhat nearer the center of the panel than assumed in Equation (70), as they were also in the Larkin Building.

If a straight mushroom slab be designed with the same quantity of concrete in it as the Donnelly slab, the drop will furnish sufficient concrete to make the slab 1.2 in. thicker, giving a total thickness of 12.2 in. The computed stress at the middle of the side belts, in case all the slab belts remain unchanged, will be $f_s = 16\,100$ lb., instead of 18 000 lb. under the test load, and the computed deflection will be barely 0.4 in. at the center of the panel.

The Donnelly slab has a 9½-ft. square frame of 1⅛-in. round rods to support the slab rods in each head, which, with the lap, is more than 40 ft. long. A standard mushroom head would require more steel than this for radial and ring rods, but the quantity is very small compared with its considerable increase of strength, and this without increasing the deflection beyond a very conservative and in every way permissible value. Therefore, a straight flat slab design, such as the mushroom, appears to be preferable to one with a large drop like this. Means for making a rational comparison not having been available heretofore, it has not been possible to state positively that the unwarranted claims that have been made for designs with a large drop were unfounded; but that such is the fact is now demonstrable, as appears from the foregoing computations.

The St. Paul Bread Company Building.

The floor of the St. Paul Bread Company Building is a rough slab, 6 in. thick, and has 16 by 15-ft. panels, reinforced with ⅜-in. round rods, ten in each belt. The design load was 100 lb. per sq. ft. The slab, Fig. 19, was constructed in winter and frozen, but, as the final test was postponed until August, 1912, the slab was very fully cured, considerably more so, in fact, than most slabs when subjected to test. The test was made by Professor W. H. Kavanaugh, of the University of Minnesota, in the following manner:

First, extensometer measurements were made on seventeen 8-in. lengths of slab rods, which were exposed under a single loaded panel, three of these, Nos. 1, 2, and 3, being at the middle of three rods of one long side belt at the edge of the load, and the remaining fourteen distributed over the central area of one diagonal belt. Second, measurements of deflections were made at two points, one at the center of the panel and one near the middle of the interior edge of one long side belt. Third, the embedment of the rods was tested. Table 2 con-

tains the observed elongations due to change of loading at all the seventeen positions for each of the test loads, 108.4, 316.8, and 416.8 lb. per sq. ft., as well as after the removal of the load. A comparison of the observed elongations at symmetrical points reveals such discrepancies in the observations as to require some preliminary discussion.

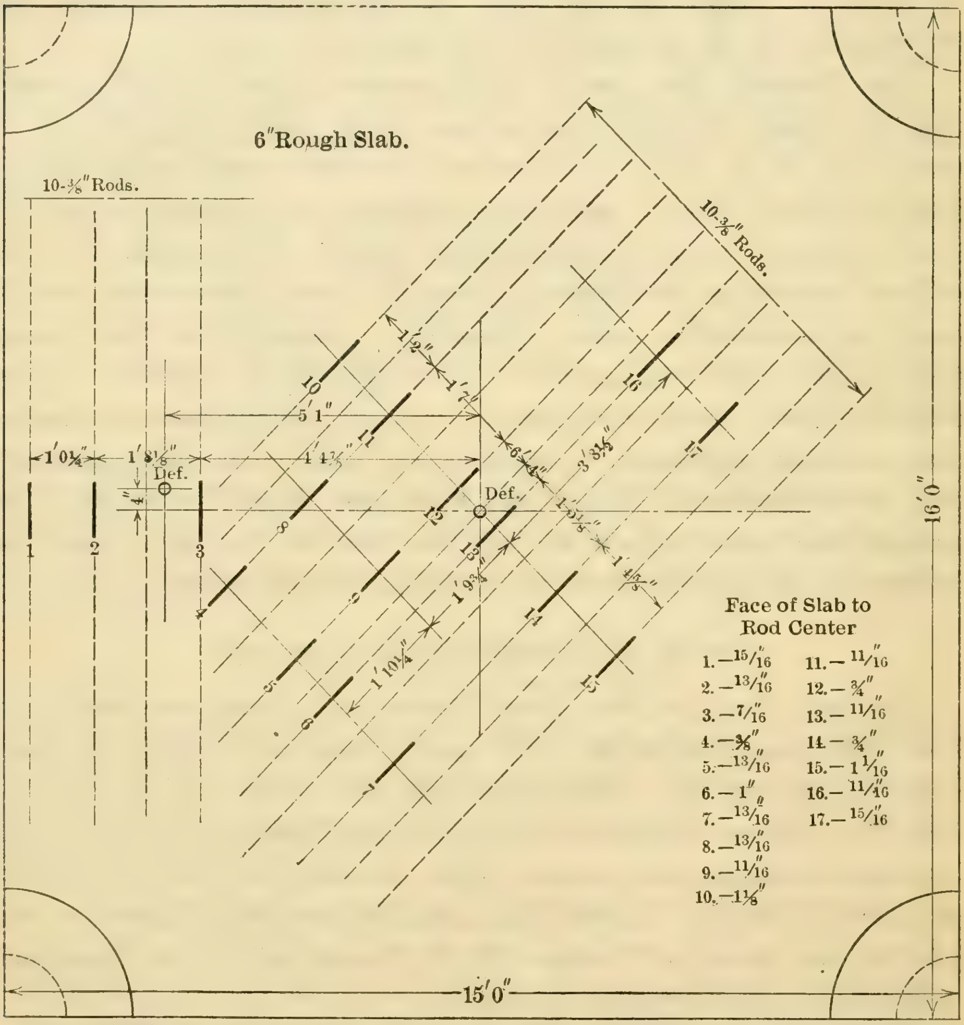


DIAGRAM SHOWING LOCATION OF TEST POINTS IN FLOOR SLAB.
SAINT PAUL BREAD COMPANY BUILDING.
"TURNER MUSHROOM SYSTEM."

FIG. 19.

Observations Nos. 5 and 6 were on a pair of diagonal rods on each side of and adjacent to the diagonal line of the panel, there being no rod exactly on the diagonal, and situated just beyond the edge of the other diagonal belt. No reason can be discerned for any difference between these elongations, but the wide

TABLE 2.—OBSERVED ELONGATIONS, IN INCHES PER MILLION INCHES, UNDER GIVEN LOADS PER SQUARE FOOT.

NOTE.—Obtain actual unit stresses by multiplying by 30.

Observation No.	ELONGATIONS UNDER THE FOLLOWING LOADS, IN POUNDS.			
	108.4	316.8	416.8	0
1	255	500	598	80
2	236	473	549	— 3
3	244	464	572	— 57
4	63	207	283	145
5	64	262	397	164
6	66	168	271	114
7	45	152	220	69
8	218	370	421	— 64
9	89	266	372	152
10	68	120	159	46
11	122	263	347	146
12	153	327	400	28
13	276	534	653	93
14	60	204	272	80
15	74	164	165	18
16	11	40	22	— 30
17	34	70	124	17

difference that appears must be due to some peculiarity in one of the rods, such as a crook or bend, or some lack of homogeneity in the concrete. Comparing Observations Nos. 5 and 6 with Nos. 4, 10, 15, and 16, which, being at about the same distance from the center, should, by Equation (49), have about the same elongations, it is found that No. 5 is abnormally large, and at the same time No. 16 is abnormally small. No. 8 is another set of abnormal results, which is evident from the fact that, being midway between Nos. 4 and 11, its elongations should lie between them; it is larger than it should be, with a final compression after removing the load. Nos. 7 and 17 should be the same, and Nos. 4 and 17 should be alike. The varying embedments of the portions of this rod which were observed show that there was probably a kink in it, which might account for the observed discrepancies. It is possible, however, that some such differences may appear when the loading is piled on one side of the panel before piling it on the other. No such explanation, however, will fit the case of Nos. 12 and 13, which are at the middle of the two rods adjacent to the diagonal line at the center of the panel. There appears to be no question that No. 13 is abnormally large, for No. 12 agrees well with others, being only a little larger than those at the nearby positions 9, 11, and 14, and the values at No. 13 are in wide disagreement with them. The

very considerable differences between results which should apparently be equal makes it evident how inexact single determinations must often be by reason of bends in the rods, lack of homogeneity in the concrete, etc., and emphasizes the importance of carefully laying belt rods straight and having them spaced uniformly, as well as embedded equally, before pouring the concrete, if consistent results are desired. It also shows the importance of checking all readings by readings at symmetrical positions.

It may be stated in general that the observed unit stresses and the deflections in this test are less than they would be for a slab tested at the stage of curing at which tests are usually made, a stage to which the equations apply more precisely. In consequence of this, all the computed results will exceed to some extent those actually observed.

Apply Equation (34) to compute the unit stress at Nos. 1, 2, and 3 of the long side belt. Assuming $d_1 = 5.3$ in.,

$$f_s = \frac{100\,000 \times 192}{175 \times 5.3 \times 1.1} = 18\,800 \text{ lb. per sq. in.}$$

in case of many panels equally loaded. The mean observed unit stresses for three rods at the edge of the load was 17 190 lb., the stress in each of the rods being practically the same, a fact that speaks well for the stiffness of the head. The computed unit stress at the center of the panel is

$$f_s = \frac{100\,000 \times 192}{256 \times 0.9 \times 5 \times 1.1} = 15\,150 \text{ lb. per sq. in.}$$

The observed unit stress at No. 12 was 12 000 lb., though at No. 13 the abnormal value of 19 600 lb. was found.

As the observations of stresses and deflections were made when only a single panel was loaded, and the computations assume that all the panels are equally loaded, any very close agreement of Table 2 with the observed results is not to be expected; nevertheless, comparison with that table shows that the computed results agree with the observations far better than the observations agree among themselves at such symmetrical points as admit of any comparisons.

The deflection at the center of the panel under a load of more than double the design load plus once the dead load, namely, 316.8 lb. per sq. ft., was 0.32 in., which is less than $\frac{1}{800}$ of the diagonal span. To

compute the deflections at the panel center, apply Equation (71), as follows:

$$D_2 = \frac{100\,000 \times 180 \times 192^2}{4.4 \times 10^{10} \times 5 \times 1.1} = 0.56 \text{ in.}$$

Computation of the deflection at the point near the middle of the interior edge of a long side belt gives a deflection for a load of 100 000 lb. on each panel of approximately 0.4 in.

TABLE 3.—DEFLECTIONS, IN INCHES, UNDER GIVEN LOADS PER SQUARE FOOT.

	TEST LOADS, IN POUNDS.			
	108.4	316.8	416.8	0
Center of panel..................Observed	0.077	0.320	0.437	0.155
" " " ....Computed	0.146	0.426	0.560	.....
Edge of side belt..................Observed	0.065	0.247	0.332	0.124
" " " " ....Computed	0.104	0.304	0.400	.....

As before stated, the somewhat large excess of computed over observed deflections is due to two circumstances: first, and principally, to the age and consequent stiffness of the slab; and secondly, at the edge, to the fact of a single panel load instead of many, neither of which circumstances is taken account of in the equations used.

THE STUDEBAKER AUTOMOBILE BUILDING.

The Studebaker Automobile Building, Michigan Avenue and 21st Street, Chicago, Ill., was erected in 1910, and is an example of a system of construction with two-way reinforcement only. The distances between column centers are 24 ft. and 23 ft. 9 in. Papers descriptive of this system may be found in the current technical journals.* All these papers, except the second, were written by T. L. Condron, M. Am. Soc. C. E., whose company designed the Studebaker Building. The last paper gives the results of tests of this and other buildings built on this system, while the second and third papers give the sizes and quantities of steel reinforcement in this particular building, from which it appears that the weight of steel reinforcement per panel was 2 600 lb.

Two kinds of floors are constructed on this two-way system; one is a perfectly flat slab of uniform thickness throughout; the other has

*_Journal_, Western Society of Engineers, December, 1909: _Engineering Record_, January 22d, 1910; _Engineering News_, January 27th, 1910; _Western Contractor_, March 12th, 1913.

a large so-called "panel" in the ceiling in the middle of the space between each four columns. The Studebaker Building is of this second kind, so that in the middle of each panel proper, 24 ft. by 23 ft. 9 in., there is this architectural "panel" about 18 ft. square. The thicker parts of the slab, extending from column head to column head, thus constitute shallow girders which in this case are 12 in. thick and about 6 ft. wide; and the central panels, 18 ft. square, are only 6 in. thick, giving a mean thickness of slab of 8.6 in., if the same quantity of concrete were uniformly distributed.

The design load was 100 lb. per sq. ft., and the dead load of the slab itself was also assumed at 100 lb. per sq. ft. The test load was applied on a single panel, and, in accordance with the Chicago Ordinances, amounted to 300 lb. per sq. ft., or a total of $300 \times 570 =$ 171 000 lb. This caused a deflection at the center of 0.34 in.

This floor will be compared with a mushroom slab, 8.6 in. thick, designed to carry the same loading.

The Studebaker slab had a thickness of 1 in. of concrete clear under the $\frac{3}{4}$-in. square girder rods, and $\frac{3}{4}$ in. clear of concrete under the $\frac{1}{2}$-in. round rods at the bottom of the central panel. The mushroom slab will be assumed to have twenty-five $\frac{5}{8}$-in. round rods in each belt, and 0.6 in. of concrete clear under the rods at the bottom of the slab, it being customary to make this fire-proofing layer less for smaller rods, and in any case not less than 0.5 in.

By Equation (34) the stress in the side belt under the dead load of 100 lb., plus the test load of 300 lb. per sq. ft., will be

$$f_s = \frac{228\,000 \times 288}{175 \times 7.8 \times 2.76} = 17\,400 \text{ lb. per sq. in.}$$

Mr. Condron states that he has used 17 000 lb. as the unit stress in the steel. Under a working load consisting of 100 lb. dead load plus 100 lb. design load the stress would be one-half the value computed above, namely, 8 700 lb. per sq. in.

The quantity of slab steel per panel in this design depends somewhat on the length of lap in the splicing. Without any lap, there would be in each 24 by 23.75-ft. panel:

Side rods,	25×24	= 600	ft.
Side rods,	25×23.75	= 594	"
Diagonal rods,	50×33.75	= 1 687.5	"

Total length of $\frac{5}{8}$-in. round rods = 2 881.5 ft.

The weight of this is $2\,881.5 \times 376 = 1\,081.5$ lb. In round numbers, it will be assumed that the steel in the mushroom head, consisting of radial and ring rods, will bring the total weight up to 1 300 lb. per panel, which is just one-half that in the Studebaker Building with the same quantity of concrete.

Now, as to deflections: by Equation (71), the central deflection due to the test load is

$$D_2 = \frac{171\,000 \times (288)^3}{4.4 \times 10^{10} \times 2.76 \times (7.4375)^2} = 0.594 \text{ in.}$$

If the cross-section of the steel were doubled throughout, the deflection would be one-half of this, or 0.297 in., which is only slightly less than 0.34 in., which was the central deflection observed in the Studebaker test.

It will be noted that the formulas in the writer's investigations are dependent on the quantity of reinforcement, and not on the directions in which it runs, the only condition being that the belts shall cross in such immediate proximity to each other as to make their mutual action on each other effective. This does not occur with perfect uniformity everywhere in the two-way system under consideration, and, consequently, it would be expected to show somewhat greater deflections with the same quantity of steel than would the mushroom system.

The special claim made for this system by its designers is that in it the stresses in the steel reinforcement can be readily computed by the method of moments due to the loading. No tests have been published, so far as the writer knows, to substantiate this claim, and in his judgment such tests would show a difference, between extensometer observations and those obtained by the formulas published in the papers previously referred to, of not less than 50% of the observed amount, and probably more nearly 100 per cent.

This conclusion is based not only on the general theory of such structures, with which the writer has some familiarity, but on the computations just given in this paper, which fairly support this statement.

APPENDIX A.

Extracts from Report of Test of the Northwestern Glass Company Building.

By F. R. McMillan.

Description of the Building.—This is a four-story-and-basement warehouse, of the skeleton type of reinforced concrete construction, erected for the Northwestern Glass Company, of Minneapolis, from plans prepared by Bertrand and Chamberlin, Architects, and Mr. C. A. P. Turner, Consulting Engineer. In plan it is 66 by 163 ft., divided into 16 by 17-ft. panels. The floor is of the "Mushroom" type of flat slab construction, designed for a live load of 400 lb. per sq. ft. The 8-in. rough slab is reinforced with fifteen ⅝-in. round rods in each direct and diagonal belt, as shown on Fig. 1. At the time of the test, the 2-in. cement finish which is to form the wearing surface was not in place.

Age and Conditions of Curing.—At the time of the test, the concrete in the test panels had been in place 5 months, but the weather had not been favorable for curing, except during the last few weeks. On the day the test panels were cast, December 6th, 1912, the mean temperature was 10°, with a maximum of 19° and a minimum of 0° Fahr. From this time until the end of March, 1913, the temperature practically remained below freezing. The usual precautions were taken to prevent freezing of the green concrete. Salamanders were kept burning in the basement until January 10th. The slab was kept covered with boards and canvas from the day it was poured until the erection of the forms for the second floor was begun. This interval was about a week or 10 days. Pouring of the second floor was begun on December 28th, and from this time until January 10th salamanders were kept fired under the second floor. The forms supporting the test panels were removed near the end of February. The last few weeks before the test the weather was very mild, and it is said that the concrete showed a noticeable increase in hardness during this period. At the time of the test, May 12th to 19th, the floor had all the appearance of an excellent hard concrete.

Plan and Description of the Test.—As the time available for the test, and the floor space that could be occupied in the basement, were limited by the necessity of caring for large shipments of glass, attention was confined to the measurement of deflections and deformations at critical points only. Very little choice was had in selecting points on the upper side of the slab at which to determine steel extensions;

for, with the type of extensometer used, measurements could only be taken on the top belt of rods and at points very near the surface.

Extensometer measurements were taken at various stages during the loading and unloading, and in all but one instance they were taken immeditely after each load was in place and again just before applying the next load. Deflections were taken at a number of intermediate stages at which no extensometer measurements were made. The extensometer measurements were made with the regular short-legged laboratory form of Berry strain gauge, reading over an 8-in. gauge line. The deflections were measured with a portable instrument specially designed to enable measurements to be taken at a large number of points without obstructing passage under the various panels.

The test load was of cement in bags, arranged in four piles in each panel, separated by aisles to prevent arching and to facilitate the taking of observations. The entire four panels were first loaded uniformly in three stages up to a total load of 540 lb. per sq. ft. of loaded area. The load from two panels was then nearly all transferred, in two stages, to the remaining two panels, making an intensity of 1 000 lb. and 750 lb. per sq. ft. on the loaded areas of these two panels. The removal of the load was made by stages in such a manner that the one panel retained its full load of 1 000 lb. per sq. ft. for 68 hours. The detail of areas and intensities of loading are shown on Figs. 5 and 6.

The accompanying tables and drawings show all the data of loading and the resulting steel stresses and deflections. These are self-explanatory. The steel stresses have been calculated from the extensometer measurements on a basis of 30 000 000 lb. per sq. in. for the modulus of elasticity. Fig. 13 shows the actual thickness of the test slabs and the depth of the rods at the top on which extensometer measurements were made.

TABLE 4.—Time Schedule of Loading.

Test of Northwestern Glass Company Building.

Date, 1913.	Load No.	Applying Load.			Interval, Load in Place.	
		Began.	Noon.	Finished.	Hr.	Min.
May 13.....	1	10:15 A. M.	12-12:30	1:10 P. M.	....	....
May 14.....	2	9:30 A. M.	12-12:30	2:45 P. M.	25	35
May 15.....	3	10:30 A. M.	12-12:30	1:50 P. M.	23	5
May 16.....	4	12:45 A. M.	....	3:00 P. M.	25	10
May 17.....	4*a*	11:00 A. M.	12-12:30	3:00 P. M.	24	0
May 17.....	5	3:45 P. M.	....	4:45 P. M.	1	45
May 18.....	6	9:45 A. M.	....	10:40 A. M.	17	55
May 18.....	7	11:15 A. M.	....	12:00 NOON.	1	20
May 19.....	8	10:30 A. M.	12-12:30	12:53 P. M.	24	53
May 19.....	9	1:30 P. M.	....	4:00 P. M.	3	7

TABLE 5.—SUMMARY OF LOADING.

Load No.	Total Panel Loads.				Load per Square Foot Over Loaded Area.			
	A	B	C	D	A	B	C	D
1	31 540	31 445	29 355	29 070	181	178	169	162
2	68 780	67 830	68 875	66 025	395	385	396	364
3	96 045	93 955	97 090	92 910	552	533	558	512
4	59 470	130 815	58 995	130 720	342	742	339	720
4a	59 470	130 815	6 745	182 970	Same.	Same.	39	1 004
5	None.	132 430	Same.	Same.	None.	751	Same.	Same.
6	"	60 135	"	"	"	341	"	"
7	"	None.	"	"	"	None.	"	"
8	"	"	"	94 715	"	"	"	521
9	"	7 000	7 000	None.	"	40	40	None.

TABLE 6.—LOADING DATA, PANEL A.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 174.1 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	A-1.	A-2.	A-3.	A-4.		A-1.	A-2.	A-3.	A-4.	Loaded area.	Whole panel.
1	8 265	9 025	7 315	6 935	31 540	182	192	178	172	181	116
2	17 385	18 525	16 815	16 055	68 780	382	394	408	397	395	253
3	24 130	25 270	23 845	22 800	96 045	531	537	579	565	552	353
4	15 105	16 150	14 440	13 775	59 470	332	344	350	341	342	219
4a	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.
5	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	"	"	"	"	"	"	"	"	"	"	"
9	"	"	"	"	"	"	"	"	"	"	"

TABLE 7.—LOADING DATA, PANEL B.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 176.3 sq. ft.

Load No.	Total Loads in Sub-Divisions:				Total load in panel.	Loads per Square Foot in Sub-Divisions:				Average Load per Square Foot of:	
	B-1.	B-2.	B-3.	B-4.		B-1.	B-2.	B-3.	B-4.	Loaded area.	Whole panel.
1	7 600	7 980	8 550	7 315	31 455	169	191	187	167	178	116
2	15 960	17 100	17 955	16 815	67 830	355	409	393	384	385	249
3	22 135	23 845	24 415	23 560	93 955	492	571	535	538	533	345
4	30 875	33 345	33 535	33 060	130 815	686	798	734	755	742	481
4a	Same.	Same.	Same.	Same.	130 815	Same.	Same.	Same.	Same	Same.	Same
5	32 490	"	"	"	132 430	722	"	"	"	751	487
6	15 485	14 820	15 390	14 440	60 135	344	355	337	330	341	221
7	None.	None.	None.	None.	None.	None.	None.	None.	None.	None.	None
8	"	"	"	"	"	"	"	"	"	"	"
9	"	3 500	3 500	"	7 000	"	84	77	"	40	26

TABLE 8.—LOADING DATA, PANEL C.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 173.9 sq. ft.

Load No.	TOTAL LOADS IN SUB-DIVISIONS:				Total load in panel.	LOADS PER SQUARE FOOT IN SUB-DIVISIONS:				AVERAGE LOAD PER SQUARE FOOT OF:	
	C-1.	C-2.	C-3.	C-4.		C-1.	C 2.	C-3.	C-4.	Loaded area.	Whole panel.
1	7 885	7 220	6 080	8 170	29 355	157	172	163	183	169	108
2	19 285	16 340	14 820	18 430	68 875	383	390	398	414	396	253
3	27 360	23 085	20 615	26 030	97 090	542	551	554	586	558	357
4	16 435	14 060	12 635	15 865	58 995	326	336	340	356	339	217
4*a*	4 750	None.	None.	1 995	6 745	94	None.	None.	45	39	25
5	Same.	"	"	Same.	Same.	Same.	"	"	Same.	Same.	Same.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	"	"	"	"	"	"	"	"	"	"	"
9	None.	3 500	3 500	None.	7 000	None.	83	94	None.	40	26

TABLE 9.—LOADING DATA, PANEL D.

Panel, 16 by 17 ft. Area, 272 sq. ft. Loaded area, 181.6 sq. ft.

Load No.	TOTAL LOADS IN SUB-DIVISIONS:				Total load in panel.	LOADS PER SQUARE FOOT IN SUB-DIVISIONS:				AVERAGE LOAD PER SQUARE FOOT OF:	
	D-1.	D-2.	D-3.	D-4.		D-1.	D-2.	D-3.	D-4.	Loaded area.	Whole panel.
1	7 410	7 315	6 840	7 505	29 070	168	158	159	156	162	107
2	16 150	16 435	16 055	17 385	66 025	365	364	372	362	364	243
3	22 895	22 895	22 515	24 605	92 910	520	494	522	514	512	342
4	32 680	31 825	31 350	34 865	130 720	740	686	728	727	720	480
4*a*	47 405	45 980	38 760	50 825	182 970	1 073	994	900	1 059	1 004	672
5	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.	Same.
6	"	"	"	"	"	"	"	"	"	"	"
7	"	"	"	"	"	"	"	"	"	"	"
8	22 895	22 895	22 515	26 400	94 705	518	494	522	550	521	348
9	None.	None.	None.	None.	None.	None.	None	None.	None.	None.	None.

TABLE 10.—Test of Northwestern Glass Company Building. Slab Deflections. Panels B and A.

Deflections are given in thousandths of an inch.

Load No.	Date, May, 1913.	Deflections at Point Number.											
		1	2	3	28	29	30	4	5	6	7	8	9
Zero.	12th	0	0	0	....	....	....	0	0	0	0	0	0
1	13th	11	22	16	....	....	....	11	15	43	32	34	18
1	14th	19	23	14	....	....	....	11	15	43	28	30	12
2	14th	59	102	60	....	....	....	44	66	105	115	121	56
2	15th	77	121	70	0	0	0	54	76	118	130	141	70
3	15th	130	206	111	86	101	69	89	124	182	219	252	135
3	16th	144	237	131	105	145	98	101	144	211	260	282	147
4	16th	158	260	138	156	251	178	94	131	186	225	241	120
4	17th	169	275	146	173	281	203	92	132	188	226	241	122
5	17th	154	250	132	170	293	211	77	102	141	160	149	71
5	18th	153	248	131	179	310	226	71	96	136	151	140	67
6	18th	....	....	....	103	187	143	....	....	....	....	....	....
7	18th	97	148	78	29	63	50	60	83	121	128	122	56
7	19th	83	139	70	20	55	40	60	82	120	129	127	49
8	19th	....	138	....	19	51	40	....	76	115	121	125	*
9	19th	89	142	74	22	62	*	50	69	105	115	116	....
9	20th	91	145	74	26	54	....	53	69	106	115	119	....
9	24th	82	136	77	17	37	....	47	63	98	104	107	....
10	June 5th	*	128	78	*	*	....	46	64	*	105	108	....

* These points were inaccessible. Note that deflections given for Points 28, 29, and 30 represent deflections below the position of the slab at Load 2. At all other points the deflections shown are from the initial position of the slab.

TABLE 11.—Test of Northwestern Glass Company Building. Slab Deflections. Panel D.

Deflections are given in thousandths of an inch.

Load No.	Date, May, 1913.	Deflections at Point Number.												
		10	11	12	13	14	15	16	17	18	19	20	21	27
Zero	12th	0	0	0	0	0	0	0	0	0	0	0	0	0
1	13th	15	25	25	32	10	17	32	37	37	27	22	11	0
1	14th	10	19	23	28	6	9	22	32	31	21	16	4	10
2	14th	49	78	104	105	42	55	79	106	109	48	78	33	47
2	15th	52	88	92	111	58	73	99	118	131	72	87	38	55
3	15th	74	124	154	166	72	95	136	168	200	95	89	50	77
3	16th	86	140	170	187	80	106	149	189	191	111	132	54	81
4	16th	92	143	179	235	94	128	183	242	251	147	137	80	89
4	17th	92	148	173	247	101	142	199	264	276	161	147	90	90
4*b*	17th	......	......	......	......	......	......	......	......	333	......	....	....	....
4*a*	17th	......	......	......	......	......	......	......	......	396	......	....	....	....
5	17th	100	164	193	343	140	198	276	387	416	250	172	150	110
5	18th	99	169	208	374	150	214	297	423	456	278	182	160	113
6	18th	......	172	211	387	154	215	308	434	466	282	188	160	116
7	18th	105	176	213	392	159	223	309	440	472	283	191	166	120
7	19th	109	185	217	415	168	237	332	474	510	304	202	174	124
8	19th	89	145	175	330	133	184	257	366	395	234	156	129	102
9	19th	57	95	101	188	76	106	149	204	216	123	93	67	61
9	20th	60	91	93	178	80	103	140	195	208	113	91	68	61
9	24th	56	83	96	147	......	94	134	176	187	101	83	61	59
10	June 5th	51	*	*	*	*	*	118	157	169	93	*	*	*

* These points were inaccessible. Note that the deflections given for Point 27 represent deflections from the position of the slab under Load 1. At all other points the deflections shown are from the initial position of the slab.

TABLE 12.—Test of Northwestern Glass Company Building.
Steel Stresses—Upper Side of Slab.
Stresses are given in thousands of pounds per square inch.

Load No.	Date, May, 1913.	Stresses at Gauge Line Number.										
		101	102	103	104	105	106	107	108	109	110	111
Zero	13th	0	0	0	0	0	0	0	0	0	0	0
1	13th	0.4	2.2	2.8	0.4	0.8	1.7	2.0	2.8	1.3	0.2	2.4
2	14th	1.5	5.8	7.1	2.0	2.7	9.7	9.3	11.6	6.4	2.0	12.0
2	15th	2.0	5.8	7.0	2.1	2.7	9.9	9.8	12.1	6.4	2.0	13.6
3	15th	2.8	7.9	9.7	2.8	2.4	14.8	15.8	18.7	10.1	2.8	20.2
3	16th	2.7	8.7	9.7	2.9	2.6	15.5	17.5	20.0	10.6	2.8	22.4
4	16th	3.0	11.2	11.2	4.5	3.7	13.8	15.5	17.8	9.8	2.1	19.6
4	17th	3.1	12.1	13.6	4.9	3.6	14.3	15.3	18.0	10.2	2.7	19.3
5	17th	3.4	16.3	18.3	7.4	5.2	10.9	10.6	13.0	9.4	2.1	14.2
5	18th	4.0	17.3	18.3	8.3	5.9	12.1	10.3	13.9	10.0	2.8	14.4
7	18th	4.2	18.1	19.3	8.5	5.8	11.4	7.6	11.9	9.8	2.8	14.4
7	19th	5.2	19.3	19.8	9.7	7.3	12.0	8.7	12.1	10.1	3.4	14.7
9	19th	3.6	5.8	6.8	4.4	5.6	8.2	7.7	8.1	5.1	0.9	11.6
9	20th	3.3	4.7	5.6	3.5	4.3	7.1	7.0	7.6	4.2	0.1	10.3

All stresses in this table are tensions.

TABLE 13.—Test of Northwestern Glass Company Building.
Steel Stresses—Under Side of Slab.
Stresses are given in thousands of pounds per square inch.

Load No.	Date, May, 1913.	Stresses at Gauge Line Number.										
		1	2	3	4	5	6	7	8	9	10	11
Zero	12th	0	0	0	0	0	0	0	0	0	0	0
1	13th	1.0	0.2	1.5	0.8	1.1	2.0	0.4	0.5	1.0	—0.4	1.5
2	14th	2.0	1.7	3.0	2.3	3.7	4.0	3.3	3.6	4.9	4.9	4.0
2	15th	1.8	1.7	2.7	1.2	2.7	4.6	2.8	3.4	4.7	5.2	4.0
3	15th	2.2	3.0	4.3	2.8	4.0	6.1	5.7	6.7	8.1	12.6	9.9
3	16th	2.5	3.4	4.6	2.8	4.3	6.4	6.5	7.5	8.2	15.0	12.1
4	16th	3.4	5.3	7.7	4.0	6.1	7.9	7.0	7.2	6.8	16.3	17.8
4	17th	3.9	5.8	8.4	4.8	6.9	8.8	7.5	7.1	7.5	17.0	19.0
5	17th	6.0	10.9	14.2	8.6	11.7	11.8	10.0	9.0	7.6	14.9	19.2
5	18th	7.1	12.2	15.7	10.3	14.2	13.5	10.9	8.8	6.9	15.1	20.5
7	18th	6.5	11.8	15.9	10.8	14.7	13.9	11.6	9.4	8.4	7.5	9.4
7	19th	7.1	12.5	17.1	11.9	16.6	15.3	12.7	10.7	8.2	6.7	9.0
9	19th	3.0	4.8	6.8	4.9	8.0	6.2	6.0	5.4	5.8	7.9	10.7
9	20th	3.1	4.4	5.9	4.4	7.1	6.1	5.5	4.3	4.5	7.9	9.6

The minus sign indicates compression. Where no sign is used the stress is a tension.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MODERN PIER CONSTRUCTION IN NEW YORK HARBOR.

Discussion.*

By Charles W. Staniford, M. Am. Soc. C. E.†

Charles W. Staniford, M. Am. Soc. C. E. (by letter).—Some of the discussion has wandered into questions not germane to the subject matter of the paper, and has entered into matters pertaining to harbor design and general methods of construction. Mr. Staniford.

The particular object of the paper was simply to show what had been accomplished by the Department of Docks in producing something more permanent than the ordinary New York wooden pier, and at about the same cost.

It is the writer's opinion, however, that all the discussions are valuable, as they bring out many admirable points in connection with harbor design and general designs for actual construction.

It is repeated that the old wooden pier, as built in New York Harbor up to 10 years ago, because it was easily removed and reconstructed, was one of the greatest factors in developing the harbor facilities. Instead of the shipping being compelled to adapt itself to the location and operation of massive permanent structures located along the water-front, as in English and Continental harbors, where there are stone quays and piers, the structures for berthing vessels and taking care of cargoes in New York Harbor were located to meet the conditions of shipping as these conditions developed and increased facilities were required. As the size of the vessels increased, the docks grew larger, in width as well as length, in proportion to the width of the slips, that is, the distance between piers increased to accommodate the greater beam of ocean steamers. It can readily be

* Continued from November, 1913, *Proceedings*.

† Author's closure.

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seen that if the New York water-front was lined with a system of massive and permanent stone, steel, or concrete structures, the development of the harbor could only proceed at enormous expense, and consequently with much less rapidity than is made possible by building structures of lighter form. The type formerly built by the Department met for the time all requirements, but, at little added expense, many objectionable features of the former construction have been eliminated, thus doing away with constant repairs; and, for the present, at least, the Department has a type which is durable and at the same time inexpensive, and admirably meets the present-day demands of shipping. The result of this more modern method of construction has been shown in the paper, and the figures substantiating the statements are based on fact, not conjecture.

Mr. Harris questions the lasting qualities of the asphalt wearing surface, and infers that a 3-in. deck sheathing would outlast the asphalt. The experience of the Department with deck sheathing is absolutely as cited in the paper, and not a matter of inference. Repairs to New York piers are absolutely needed at short intervals, resulting in frequent renewal of the whole deck sheathing, the wear on the latter being the result of a traffic movement probably unequaled in the world. In fact, this whole paper deals with conditions of this kind, where structures are under intense use constantly, and not with docks built in Navy Yards or in any place for public function, where team traffic is almost unknown, or where the docks are practically intended as a berthing place for steamers while undergoing repairs.

As a concrete illustration of the inexpensiveness of the asphalt wearing surface, the piers of the Chelsea Section may be cited. On these nine piers, each approximately 800 ft. long and 125 ft. wide, asphalt wearing surfaces were laid, and for a period of 5 years, under the most extreme traffic conditions, no repairs were necessary. Under the contract for this work, 96 989 sq. yd. were laid, and a bond for 5 years was given to maintain the asphalt wearing surface; at the expiration of this 5-year bond period, repairs were only needed on 780 sq. yd., costing $800, or less than 1% of the total cost. From the expiration of the bond period to the present day no repairs of these surfaces have been necessary; and, furthermore, no repairs are needed at the present time. Fourteen other piers may be cited where the same result has been secured by the use of asphalt.

As stated in the paper, for light traffic, this asphalt wearing surface may be eliminated, but, for heavy traffic, it has been found absolutely necessary to put some deck wearing surface over the concrete; and in some other localities the writer has advocated wood block or brick pavement for this purpose. Granite block pavement has been considered, but was not adopted because it would add so much to the weight.

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Mr. Harris questions the facility of making repairs to decks of this kind. These repairs, like all others relating to dock and harbor structures, must be made by specialists in work of this class. The Department force, consisting of twenty pile-drivers with their crews, is constantly engaged in doing just such work.

TABLE 4.—PIERS ON WHICH ASPHALT SURFACES HAVE BEEN ADOPTED.

Location.	Area, in square yards.	Price per square yard.	Elapsed time, in months.	Area repaired, in square yards.	Percentage of area, in stated time.	Percentage of area repaired per year.
Piers 53 to 62, N. R.	96 989	$0.90	34	40	....	....
		....	3	70	....	....
		....	3	175	....	....
		....	13	280	....	....
		....	2	145	....	....
		....	5	70	....	....
		Total:	60	780	0.8	0.16
ier, W. 40th Street, N. R.	2 817	$1.48	48	none.	....	....
" 5, E. R.	4 315	1.19	30	none.	....	....
" 6, E. R.	3 043	1.19	12	25	....	....
			9	50	....	....
		Total:	21	75	2.5	1.43
Pier 22, E. R.	2 785	$1.48	60	20	0.7	0.14
" E. 5th Street, E. R.	2 876	1.18	22	25	1.0	0.55
" E. 20th Street, E. R.	3 602	1.49	46	70	2.0	0.52
" E. 95th Street, E. R.	1 614	0.90	27	none.	....	....
" E. 100th Street, E. R.	1 095	0.90	28	none.	....	....
" E. 110th Street, H. R.	1 964	1.49	48	none.	....	....
" Canal Street, Stapleton.	2 296	1.23	60	20	1.0	0.20
Platform, Stapleton Ferry	1 594	2.40	60	35	2.2	0.22
Pier, 52d Street, Brooklyn	2 719	0.84	23	25	1.0	0.52
" Whale Creek, Brooklyn	4 788	0.98	16	none.	....	....
" Jamaica Avenue, Queens.	742	1.45	18	none.	....	....

It is mentioned that the eventual destruction of caps and stringers by continuous repairs deserves consideration. This is just what was considered in the design of piers of this style. By eliminating all the stringers and all the deck, leaving practically little more than the timber caps, the quantity of timber to be renewed is reduced to an absolute minimum, the life of the caps, in addition being prolonged considerably by the use of the new system with its water-proof deck. In time, when repairs to this composite structure are required, owing to the rot of this insignificant quantity of timber underneath, such repairs will not necessitate the disturbance of the concrete deck, as all the work may be done below it by simply supporting the structure between alternate pile rows, removing the caps, and bench-capping

Mr. Staniford. the piles, the usual method by which similar repairs have been made by the Department for 25 years or more.

The repair of an occasional pile in the interior of the pier, on account of rot, is accomplished without any trouble by cutting the pile and replacing the head by bench-capping or splicing, thus making the structure as strong as the original. This has been done in a number of instances in New York, even under elaborate buildings, such as ferry houses, without any trouble whatever, and without any interruption of traffic.

The design of this pier offers relief from constant repairs, and makes unnecessary the expense of actual reconstruction from mean low water up, for the experience with the asphalt deck surface has covered a sufficient time to prove that repairs have been greatly eliminated by this feature, and the structure is actually preserved by the asphalt deck surface.

The writer gave what he considered to be full recognition to the design of the piers being built in the Brooklyn Navy Yard, as being a good type for producing an absolutely permanent structure in these waters. To produce anything in the line of permanent development, however, is simply a question of expense. Mr. Harris has given some figures relative to the cost of piers in South Brooklyn, and has inferred that these prices were not a fair criterion, it being contended that the piers were built at extremely low cost, and with no profit to the contractor. This contention is difficult to prove. In the paper, the cost data for these piers are correct, and are repeated and elaborated as follows: The 31st Street Pier was let to the lowest bidder at $174 587. This was one of twelve bids, the next four higher being: $183 820, $192 462, $195 897, and $196 725. The 33d Street Pier was let to the lowest bidder at $236 500. This was one of seven bids, the next three higher being: $244 587, $246 332, and $254 897.

In each case, the bidders for these piers were among the most reliable dock-building contractors in New York; and without question, any one of them could have performed the work successfully for the amount of his bid.

The New York Department of Docks has developed many designs for absolutely permanent structures, not only on the lines of the piers in the Brooklyn Navy Yard, made permanent from low water up, but absolutely permanent structures with concrete piles, and various forms of filled-in piers, some of which are without doubt as good as those cited. In any event, no two designs are exactly the same. Where the money is available, and the cost of deck repairs will necessarily be slight, and where a permanent structure is warranted, Mr. Harris is unquestionably right in suggesting that the structure should be made permanent from low water up.

Mr. Staniford.

Mr. Harris states that the generally accepted opinion that there are no marine borers in New York Harbor is in error. He states that they are found in the Lower Harbor, and that perhaps under certain conditions they may develop with wind and tide on the South Brooklyn shore. The writer fails to see why he cites the fact that he had found sea borers at Fort Lafayette. This question of the presence of the teredo, limnoria, or other wood-boring animals was settled by the Department more than 40 years ago. In fact, it was not only settled by the Department, but by all the corporate interests which have helped to build up New York Harbor while spending several hundred millions of dollars in building wooden structures. Unquestionably, if New York had remained a settlement with none of the evils attendant on the growth of a city of even ordinary size, these wood borers would have remained in the harbor, and the present form of construction would not have obtained. This has not been the fact, however, and 30 years ago the last evidences of the activity of these animals disappeared in all pier construction built either by the City or by others in the Upper Harbor.

In presenting this pier as a type of construction for New York Harbor, the writer took it for granted that it would be understood as applying to piers to be built in the harbor proper, and not for outlying sections nearer the ocean, where, of course, the teredo exists, this form of sea life requiring clear and fresh salt water in order to live. These wood borers have long been driven from the harbor proper. Even 10 years ago, the Department, in approaching the zone of danger (at the end of the Lower Bay) adopted the plan of creosoting all piles, but even in such localities, namely, along the northerly shores of Staten Island, at St. George, and other places where the teredo had been somewhat active, untreated piles which had been in the water for 8 years have been pulled and have shown absolutely no sign of any wood borers.

Before determining to use timber in the South Brooklyn Section, the Department for a number of years made a most exhaustive investigation, with divers, of hundreds of existing piers, bulkheads, and other structures along the water-front, looking for signs of infection by the teredo. The result showed that, although this sea worm had been somewhat active there in years past, it is now practically extinct.

Furthermore, for years past, the Department has been continually repairing the older structures built by the City. In the course of these repairs, the renewal of piles, even in the older structures, is found necessary only at rare intervals in the interior of the piers where they have been protected by the timber deck; though near the sides, where exposure is greater, more extensive repairs are needed. Also, the Department is continually removing old structures, some of them the oldest in New York Harbor, to make way for improvements. In these

Mr. Staniford. removals every pile is examined, under orders, for any evidence of the teredo, in order to obtain some idea of the time when it disappeared from that particular locality. In some instances the piles show that they were attacked by the teredo in the olden days, but, probably on account of increasing pollution of the water, it had ceased to exist. Piles removed from structures built within the past 25 years are as good as the day they were driven, and are constantly used again in other work.

Recently, the Department has been removing some of the old Brooklyn ferry houses on the East River, these structures being among the very oldest on the water-front. In pulling the piles, each was examined with particular care to see if the teredo had made any ravages, but no evidence was found in any case.

The following is a concrete example of the wonderful preservation of these piles and the non-existence of the teredo. When the old Pennsylvania Ferry House, at the foot of Cortlandt Street, one of the oldest structures along the lower water-front near the Battery, was being removed, the writer visited the premises in order to determine whether it would be advisable to use some of the old piles, which happened to be in the right position to take their place as part of the grillage for the concrete retaining wall which was to run under the new ferry house. He found that the piles, cut several feet above low water, were in a remarkably good state of preservation; the foreman, on being told that the piles were as good as the day they were placed, replied, "They are better than the day they were driven." They were subsequently retained and remained in the permanent structure as specimens of absolutely perfect piles, thus saving the cost of placing a number of new ones.

Mr. Snow says:

"There are many examples of concrete construction in sea water in Boston Harbor, and in every instance known to the speaker there has been serious disintegration between high and low tides."

In New York this condition does not obtain in the concrete works produced by the Department of Docks. A few other concrete bulkheads have disintegrated in this way, notably at the Brooklyn Navy Yard, where such a bulkhead, approximately a mile in length, has entirely disintegrated and been removed. The last piece of this concrete wall is now being demolished near the end of the Cob Dock, where absolute disintegration has taken place throughout, extending many feet back from the face, including the front piles, ends of caps, etc.

It is presumed that the Government will replace this wall with a new concrete structure, and then all the old Navy Yard concrete wall will have been reconstructed.

Up to the present time there has been no failure in any part of more than 8½ miles of concrete and granite wall construction, built

by the Department of Docks and Ferries, during a period of more than 40 years. Mr. Staniford.

Frank W. Hodgdon, M. Am. Soc. C. E., Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners, writes as follows:*

"Practically all the structures built of concrete in Boston Harbor have been badly disintegrated between high and low-water marks."

To this the writer replied as follows:†

"That it will be of interest to your readers is evidenced by the fact that this office is in continual receipt of letters of inquiry from all parts of the world, not only from harbor engineers, but from engineers engaged in the design of bridges in order to ascertain the condition of such work built of concrete in salt water, in view of the failure of such structures throughout the world.

"It certainly is a question concerning which engineers should receive all information possible whereby people at large may receive due consideration in assuming the financial responsibility in future for constructions of this kind which are supposed to be built with a view to permanency.

"I will not attempt to give the history of anything but city construction work in New York Harbor, and the condition of same at the present time. Most of the work done here during the past forty years has been done by the Department of Docks, or what is now known as the Department of Docks and Ferries of the City of New York. During this period the city has been engaged in the construction of a permanent sea-wall, located principally around lower Manhattan, below 80th Street. This has been added to recently by quite extensive construction at North Brother Island, in the Borough of the Bronx, at Whale Creek in the Borough of Brooklyn, and at the Gowanus Section, South Brooklyn.

"This sea-wall in a general way varies in type of construction, according to the character of foundation, but the essential features above the foundation are very similar in all types. Almost every type of bottom is encountered in New York Harbor, while the wall has been built to meet these varying conditions, about twenty-five different modifications having been used thus far in the foundation work to meet the conditions imposed. There are three general types, however; one built upon solid rock, one built upon hard bottom, such as good, stiff clay or compact sand, and one built on soft river mud, forming a wall supported by mud flotation.

"In many parts of the waterfront of Manhattan, for instance, bedrock is over 200 ft. below the river bottom, overlaid with soft mud. In the rock sections the wall has acquired a height above the rock as much as 50 ft. In other rock sections the wall is founded upon rock that has had to be blasted in order to create a sufficient depth for wharfage in front, and to take off sufficient rock for a stepped-off bottom in order to safely found the wall against the slope of the natural rock.

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* *Engineering Record*, June 10th, 1911, p. 655.

† *Engineering Record*, August 5th, 1911, p. 176.

Mr. Staniford. "In the soft mud bottoms the longest piles are used. In most types requiring pile foundations, the piles are usually cut off at a depth of about 15 ft., below mean low water, and from the top of these piles the wall proper is built.

"In the case of the rock bottom type the wall proper may be considered to commence at the top of the bag foundation which is built immediately on the rock bottom to a proper grade, for receiving the wall proper. From the tops of these pile and concrete bag foundations, the wall is built up to the vicinity of mean low water of concrete blocks, varying according to design from about 20 to about 95 tons in weight. These concrete blocks are moulded in air in mould boxes and are shipped on scows from the yard at which they are fabricated to any point on the waterfront where they are to be used. For handling these blocks we use two derricks, according to weight of block, a 40-ton derrick and a 100-ton derrick.

"Above these blocks the type of construction is generally the same, consisting of four courses of granite facing, of headers and stretchers, dove-tailed into a thick backing of concrete deposited en masse in the rear of the granite, and all carried up to the grade for the granite coping. In several places concrete deposited in situ forming a homogeneous mass with the concrete backing is substituted for granite and carried up to the coping course. This is usually done at points particularly where piers project from the bulkhead, as this portion of the wall is covered by the pier and, therefore, not visible. This type of construction with concrete face, instead of granite face, appears also in the general construction of the Claremont Section, the 116th Street Section, the Gowanus Section in South Brooklyn, the Whale Creek Section in Brooklyn, the Fulton Section on the lower East River, and part of the Old Slip Section on the East River.

"All of this wall construction has been built in the river itself, generally well off from the shore, in salt water, in a tide with an average range of 5 ft., varying in extremes to 8 ft. The tidal and current conditions in most parts of the East River waterfront are so severe that a diver is not able to stand up and work for more than half an hour in slack water, necessitating the construction of temporary breakwater barriers as a protection for the diver and for work against the current.

"The Department of Docks and Ferries, up to the present time, has built about 8½ miles of this river or sea-wall, requiring in its construction in the neighborhood of 6 000 concrete blocks. In the course of years the Department has had many opportunities for examining the condition of this wall in connection with the building of sewers, etc. Up to the present time no disintegration has been discovered that can be attributed to the existence of the structure in salt water. The concrete itself is in an admirable state of preservation, absolutely hard, and is undergoing no regular process of disintegration. In some cases between high and low water marks, where the concrete has been deposited in situ, there are indications of ice abrasions, but nothing whatever to cause any softness or to expect ultimate trouble.

"In some cases there are indications of the effect of wave action or scour on the freshly deposited concrete within the tidal range, due

CONCRETE BLOCKS BUILT AND PLACED IN WALL BY DEPARTMENT OF DOCKS, AND REMOVED BY THE EDISON COMPANY IN THE CONSTRUCTION OF INTAKE TUNNELS.

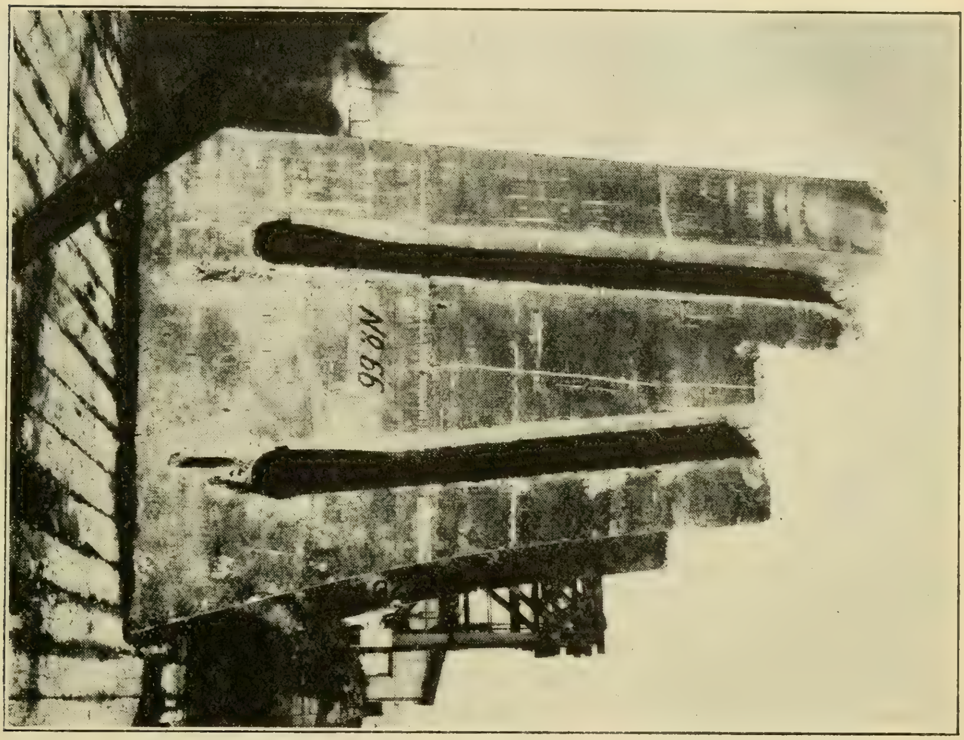

FIG. 13.—END VIEW.

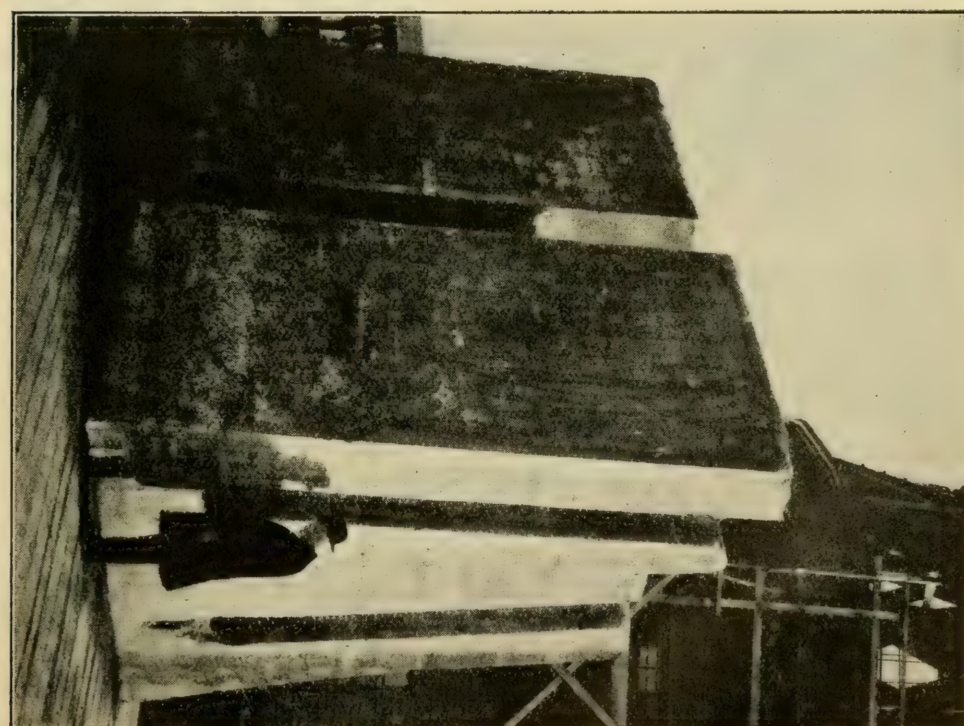

FIG. 14.—BACK VIEW.

to carelessness in work on the mould boards, creating opportunity for tidal or wave scour, but this trouble does not appear to be progressive after the original scouring action on the freshly deposited concrete, excepting in the case of gravel concrete. It is my opinion that gravel concrete is not a proper material to use en masse (not moulded in air), where it is to be subjected to wave action or tidal effect in the process of depositing. Mr. Staniford.

"Up to the present time the Department has not been obliged to spend much money on repairs to the sea-wall. This wall was built to be permanent, and at the present day all of the exposed concrete, which in the course of a few years may become a little worse from abrasion than at the present time, can be repaired and brought to its original state with the expenditure of a few hundred dollars.

"The extreme conditions that exist in Boston and that are to be seen in certain parts of this harbor, outside of work done by the city, and, in fact, in all of the various harbors of the world, could not exist and continue in the municipal work of the City of New York under its present financial control and government.

"Concrete harbor works built in other cities under municipal, private or corporate control, as well as constructions in European harbors, were, of course, all built originally as improvements, at great expense, over timber construction which might have been built under ordinary expenditure at a far less cost, and are only permanent investments, according to their behavior; therefore, because of the partial failure or threatened failure of so much concrete work in harbor construction, leading engineers throughout the world are assuming unusually expensive construction in their endeavors to produce permanency, by going to the other extreme and using granite or stone.

"Notwithstanding the failure or partial failure of concrete in the presence of salt water, in certain localities, the fact remains that this sea-wall which has been under construction by the City of New York for 41 years, is at the present time an excellent piece of work and is subject to the same climatic conditions as all cities on the Northern Atlantic Coast with the attendant ice, cold and rain characteristic of this latitude."

With reference to the concrete blocks mentioned by Mr. Betts as having been removed from the bulkhead wall after immersion in the salt water of the harbor for 10 years, Figs. 13 and 14 are submitted. These are from photographs of the blocks after they had been removed from the wall at 38th Street, East River, and placed on the Department Dock at the East 24th Street Yard. These blocks, weighing about 80 tons each, and recorded as Nos. 66 and 82, respectively, were fabricated on June 20th and August 16th 1902, and were set in the East 38th Street bulkhead wall on July 14th and August 30th, 1902. They were removed from the wall by the Edison Company, in the construction of intake tunnels, on October 31st, 1912.

The abrasions along the top and parts of the bottom edges of these blocks are the result of chiseling, in taking the blocks out of the wall and breaking their contact with the concrete backing and

Mr. Staniford. granite which surmounted them. They are in a perfect state of preservation, showing no disintegration, even to the extent of the displacement of a particle. The edges are practically knife-edges, and the sharp-cut ridges along the joint lines of the mould boards show clearly. These blocks will be used in other work of the Dock Department when convenient.

Mr. Taft has presented a design for a permanent concrete dock which no doubt would answer for some localities where the bottom would permit of constructing a foundation such as he recommends. It is novel in many of its features, and has many good points; but the question of decreasing the thickness of slab in the type of deck now being built by this Department would not be considered here. This is a question that cannot be decided analytically, but must be answered by experience with structures subject to the heavy side shocks which piers receive, together with the shocks from traffic, and from merchandise falling while being handled on the pier. The slab described in the paper has been tried out and its thickness is slightly greater than that used in the original piers; any future change, if ever made, would be to increase it.

It may be interesting to many of the members of the Society to mention that another governing principle with the Dock Department engineers, in evolving a type of pier leading to the use of a reinforced concrete deck, was the elimination of any possible patented process. Municipal contracts in New York must be prepared so as to avoid the use of any patented processes, in order that competitive bids may be received to produce the structure under the design presented. This is not only a barrier, but is troublesome throughout the whole preparation of the contract and while the work is under construction. The Department, therefore, has evolved a type which, at least, has not caused any trouble from this source; it simply calls for a prescribed size of rod and spacing, or their equivalents in other shapes, on which a bid can be made by anybody manufacturing steel for reinforced concrete.

Mr. Taft's type, however, has many good features, and might well be used by private corporations desiring a permanent structure, the writer's only objection to it being the thin slab for heavy traffic. This is not such a serious factor as to detract from its usefulness, as the necessary increase in the thickness of this slab would not add greatly to the cost, although Mr. Taft makes quite a feature of this.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

FLOOD FLOWS.

Discussion*

By Robert E. Horton, M. Am. Soc. C. E.

Robert E. Horton, M. Am. Soc. C. E. (by letter).—On page 1057,† Mr. Fuller states that: Mr. Horton.

"As far as the writer knows, the first suggestion of the use of the element of time in a formula for flood flows was made by Allen Hazen, M. Am. Soc. C. E., in a memorandum written by him in 1910."

It is not disputed that the idea of the use of the element of time in the discussion of flood flows may have occurred to Mr. Hazen independently, but in deference to the late George W. Rafter, M. Am. Soc. C. E., the writer feels obliged to deny that the idea originated with Mr. Hazen. This idea, as to its application to floods and also to other hydrological data, was suggested to the writer at least as early as 1896, or 14 years before it was suggested to Mr. Fuller by Mr. Hazen.

It was Mr. Rafter's desire that the writer should prepare the data for a paper on the application of the Theory of Probability to Flood Flows and other hydrological data, and the matter was discussed frequently and fully during the years 1896 to 1904. It appeared at that time, however, that stream flow data were so meager as to make it desirable to defer the completion of the studies. The writer continued the studies, as occasion arose, using the ordinary law of probability where the data were too meager to permit of plotting a satisfactory special probability curve on logarithmic paper.

* Continued from December, 1913, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., May, 1913.

Mr. Horton.

In 1896 the writer had prepared, at Mr. Rafter's request (by letter dated October 25th, 1896), an analysis of the rainfall and run-off of the Upper Hudson Drainage Basin, in which the probable average interval of recurrence of run-off less than the observed minimum was determined by the Theory of Probability. This and similar phenomena were analyzed by the ordinary method of probability, through the use of the so-called Gaussian Law of Error.*

It was recognized at that time that the Gaussian Law did not represent accurately the law of frequency of recurrence of many hydrological events, and an effort was made first to find a better general law† and to deduce individual laws. For the latter, the method of plotting flood records on logarithmic paper was tried, and proved so promising that, between 1900 and 1908, many logarithmic probability curves were plotted by the writer and corresponding formulas deduced, involving time as an element; and the methods and results have been applied extensively in professional practice by the writer for a period of several years.

He also applied the method of analysis of flood data by logarithmic special probability curves to the determination of maximum navigable stages, capacity of waste-weirs, etc., in connection with his work as Resident Engineer in Charge of the Bureau of Hydraulics of the New York State Barge Canal.

These results, after discussion with other engineers of the Department and with the Advisory Board of Consulting Engineers, were used as a basis in the design of work costing some millions of dollars. In this work the writer adopted 100 years in most cases as the average interval of recurrence of a flood of the magnitude to be made the basis of design. For the determination of relative frequencies of floods of different magnitudes on small torrential drainage basins in the Upper Mohawk River area, for which flood records were mostly wanting, the writer used as a basis of comparison in 1906 logarithmic flood discharge formulas which he had previously deduced for the Neshaminy, Tohickon and Perkiomen, as follows:

$$Q = 30\ T^{0.25} \text{ for the Neshaminy,}$$
$$Q = 30\ T^{0.27} \text{ `` `` Perkiomen,}$$
$$Q = 40\ T^{0.25} \text{ `` `` Tohickon,}$$

in which Q = flood discharge equalled or exceeded in an average interval of T years, in cubic feet per second per square mile.

These formulas were based on 20 years' records of the streams, covering the period, 1885-94. They give the results shown in Table 37.

* Report of State Engineer and Surveyor of New York, 1896, p. 841 *et seq.*

† In a letter dated August 11th, 1897, Mansfield Merriman, M. Am. Soc. C. E., suggested to the writer the use of a function of the form $y = (x + a)^2\ (x - b)^2$ for this purpose.

The writer also deduced the following general formula, applicable to these and similar drainage basins: Mr. Horton.

$$Q = 4\,021.5 \frac{T^{\frac{1}{4}}}{A},\ T = \sqrt[4]{\frac{Q\,A}{4\,021.5}}$$

or, as a logarithmic formula,

$$\log.\ Q = 3.6043881 + \frac{1}{4} \log.\ T - \log.\ A,$$

in which A = drainage area, in square miles; other notations as in Table 37.

TABLE 37.

Stream.	Drainage area, in square miles.	Interval, T, in Years.				
		1	10	25	50	100
		Cubic feet per second per square mile.				
Perkiomen	152	29.7	49.6	60.5	70.5	82
Neshaminy	139.3	29.4	55.0	71.0	85.5	104
Tohickon	102.2	40.5	71.0	89.0	105.0	125

In view of the foregoing considerations, the writer fails to discover wherein there is any proper basis for a claim of originality for this method, as far as the use of time as an element in flood formulas is concerned, as set forth in the paper. The writer does not claim any great breadth of applicability for the general formula just given, but believes that factors other than area modify the flood discharge of streams so profoundly that it is better, wherever possible, to derive individual formulas or flood-frequency diagrams for each stream. Following this idea, he deduced flood-frequency diagrams in 1906-08, for use in connection with the New York Barge Canal work, for the Mohawk, Hudson, Oswego, Seneca, and other streams. He has deduced and used similar special probability curves and logarithmic formulas to determine the law of frequency of recurrence of various other hydrological events, such as minimum seasonal rainfall, duration of periods without rain, relative probability of floods in different months of the year, minimum run-off of streams, storage volume required to maintain a constant supply from a stream, rain intensity of various durations, etc.

Attention may properly be called to another study, utilizing the same principles, which seems to antedate Mr. Hazen's suggestion of 1910, namely, the rain intensity diagram and formulas of J. de Bruyn-Kops, M. Am. Soc. C. E., for Savannah, Ga.*

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 248.

Mr. Horton.

With reference to the Neshaminy, Perkiomen and Tohickon flood formulas derived by the writer in 1904, as given in Table 37, it should be noted that these were derived in such a manner as to include all floods exceeding a given magnitude, and not merely the greatest single day's flood of each year. The writer's usual method of deriving a formula for the frequency of annual floods which will equal or exceed a given limit is similar to that described in Table 5, Part *c*, except that in his analysis Mr. Fuller discards all floods less than the mean, and works from ratios. The writer utilizes all annual floods, both greater than and less than the mean, although it is sometimes necessary to use a different flood formula for floods of low intensity. He also prefers to work directly from the original data rather than by reducing the data to the form of ratios.

Mr. Fuller says of his general formula, as given on page 1014:*

"$Q = CA^{0.8}$ $(1 + 0.8 \log. T)$, in which Q is the largest 24-hour average rate of flow to be expected in a period of T years."

Tracing out the derivation of this formula (given on pages 1023-1027*), it appears that Q is there defined as the "average maximum flood," and the meaning of this term appears to be quite different from that first given.

Assuming that Mr. Fuller's formula represents the average magnitude of floods having an average interval of recurrence greater than T, it may also be said that it represents the magnitude of a flood which will be equalled or exceeded at some average interval greater than T, but that this interval is unknown and cannot be determined from his formula. This illustrates the indefiniteness of his formulas when applied to the problem of flood determination, in the form in which, from the writer's experience, this problem usually occurs.

Again, in the case of Tohickon Creek, Mr. Fuller apparently does not mean by his formula for the average maximum flood of that stream, $Q = Q$ (Ave.) $(1 + 0.76 \log. T)$, that we may expect a flood at least as great as, or in other words, equal to or greater than 6 300 cu. ft. per sec. once in 5 years, for his own analysis of the same data (Table 5, Part *c*) shows that we may expect a flood to equal or exceed 6 300 cu. ft. per sec. on an average only once in about 10 years, or one-half as often.

On reading the paper carefully, the writer is of the opinion that the author has failed to set forth clearly the meaning of his own deductions. The writer's understanding of the matter is as follows, referring to Table 5, Part *a*, for Tohickon Creek.

The average magnitude of maximum floods having average intervals of recurrence between 5 and 25 years is 1.53 times the mean flood, or 6 300 cu. ft. per sec.

**Proceedings*, Am. Soc. C. E., May, 1913.

Mr. Horton.

The average magnitude of floods having intervals of recurrence between 2.50 and 25 years is 1.29 times the mean flood, or 5 310 cu. ft. per sec., and so on.

The results of the author's general formula bear the same interpretation, except that it is probably approximately true, from the method of analysis used, that Q, in the formula, represents approximately the average volume of all floods having an average interval of recurrence greater than T, it being here assumed that the formula is correct for the particular stream in question.

What Mr. Fuller has really deduced is not the average magnitude of the floods having, for example, 25 years or 12.5 years average interval of occurrence. Statistically speaking, there can be only one magnitude of the maximum flood having for example 25 or 12.5 years average interval of recurrence. The appellation "average maximum flood" of a given interval of recurrence seems to be misleading and inaccurate. If we had a long enough record, and were to divide it up arbitrarily into 25-year periods, and were to take the average of the maximum floods of each period, we would get a quantity having some semblance to a maximum average flood of 25 years interval, but the writer fails to see any practical value in such a result. Furthermore, the result would obviously be very likely to be affected by the mode of dividing up the record, which condemns the method.

Take, for example, the 5-year Tohickon flood of 6 290 cu. ft. per sec. The author does not mean that we may expect a flood of just this magnitude on an average of once in 5 years. As a matter of fact, the probable average interval of recurrence of a flood of just this magnitude is very much greater than once in 5 years.

The writer cannot see that results in the form of the "average maximum flood" are of much practical use, but, if any one wants them, they are derived more easily and directly by the method shown in Table 5, Part *c*, than by the method used by the author.

In addition, it would seem that the method of Table 5, Part *c*, when properly applied, has several other decided advantages.

1.—It is simpler of application, requiring three columns instead of five.

2.—It gives directly the average interval of recurrence of a flood equal to or greater than a given magnitude. This is a factor of great importance, and has direct application in designing spillways, determining high navigable stages of waterways, etc.

3.—It enables the average interval of recurrence of a flood lying between any two given magnitudes to be readily determined.

For example, from Table 5, Part *c*, the average interval of Tohickon floods at least 1.21 times the mean, or 4 968 cu. ft. per sec., is five years. Their probability is, therefore, $\frac{1}{5} = \frac{5}{25}$.

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The average interval of floods of at least 1.45 times the mean, or 5 958 cu. ft. per sec., is 8.33 years. Their probability is, therefore, $\frac{1}{8.33} = \frac{3}{25}$. The probability of a flood between 4 968 and 5 958 cu. ft. per sec. is, therefore, $\frac{5}{25} - \frac{3}{25} = \frac{2}{25} = 0.080$, or the probable average interval of recurrence of a flood between 4 968 and 5 958 cu. ft. per sec., or of a flood having an average of about 5 460 cu. ft. per sec., is 12.33 years. Now, if Table 5, Part *a,* and the formulas deduced by the method there given, mean what the author says they do, would we not expect floods of this average magnitude to occur at intervals of about 3.57 years, instead of 12.33 years, as actual experience shows to be the case? This illustrates what seems to be a fatal error in the author's method of analysis.

It would appear that the relation between flood magnitude and frequency, when expressed in terms of either "average maximum flood" or "median flood," is not only very indefinite and difficult of comprehension, but is apt to be very misleading, if it is not indeed practically meaningless. It does not directly convey the information which the engineer usually most desires, for example: If a spillway has a capacity of 1 000 cu. ft. per sec., how often on an average will its capacity be exceeded?

Further, in Tables 4 and 5 for Tohickon Creek, the author has arbitrarily excluded from consideration all floods which did not exceed the average yearly one of 4 117 cu. ft. per sec. He has excluded thirteen floods and used eleven. Instead of following the rule "of making the best possible use of all the available information", he has made no direct use of considerably more than half of it. What the author was seeking was, it may be assumed, to find the law defining the relation between the magnitude of floods and their relative frequency of recurrence. If one were seeking to determine the law of velocity of falling bodies, and had 25 observations of the time required for different weights, varying from 10 to 60 lb., to fall through given heights, it would hardly be considered good scientific method to discard *a priori* all observations on weights of less than 35 lb.

Mr. Fuller's method of working with ratios (Table 5), instead of actual flood volumes, seems to be confusing and cumbersome. The data which the engineer has to start with are actual flood volumes; what he desires to determine, as a rule, as in designing a spillway, is a flood volume which will not be equalled or exceeded on an average oftener than once in a given interval, say, 100 years.

If we had two drainage basins, *A* and *B,* which were similar figures, with similar drainage systems, or "stream nets" as the German hydrologist calls them, and identical in geology, soil, topography, slope,

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culture, and climate, but if A were larger than B, and if Q_A and Q_B were the true maximum 24-hour floods to be expected on these streams in an average interval of T years, then the difference between Q_A and Q_B would be the difference in flood-yielding capacity due to area alone.

In practice, however, it often happens that one area much larger than another does not yield a materially larger flood. If the data of such heterogeneous basins are plotted together in terms of area as abscissas, it appears to the writer that the results will be so discordant as to render the conclusions drawn therefrom of doubtful value.

Here we come squarely against the question of the physical, and not merely the statistical, aspects of the question. The latter only are covered by Mr. Fuller's discussion.

In the climate of the North Atlantic States local thunder storms nearly always produce maximum floods on very small areas, though only broad general cyclonic storms produce floods on the larger and medium large rivers, like the Hudson, the Susquehanna, and the Delaware. Now, thunder storms produce rains of great intensity but short duration. Broad cyclonic storms produce rains of low intensity but long duration, so that the relative flood intensity for different areas is not wholly determined by the relative times required for the water to reach the stream, or by the physiographic or cultural conditions, but partly by purely meteorological conditions.

The paper contains a valuable collection of flood data. It would have been easy to include the dates of the maximum floods, and these would be of use to engineers in comparing simultaneous flood intensities at different places. The writer cannot help feeling a sense of regret that so much labor has been expended in analysis of flood data along lines that do not seem to be either the most scientific or the most useful to engineers.

The ambition to place the results of one's investigations in the hands of the Profession at the earliest date is laudable, if indulged in with moderation. The writer's only excuse for withholding from publication the results of his own studies along similar lines for so long a time is that he thought it better to try the method out in practice first, rather than run the chance of punishing the Profession by the publication of that which might later prove to be premature.

The writer happens to have collected all the data contained in Table 13. Many of the floods there listed have subsequently been exceeded.*

* Reports on Hydrography (by the writer), State Engineer's Reports, 1900, 1912; also paper on "Effects of Recent Flood on New York Streams," *Engineering Record*, April 12th, 1913.

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PAPERS AND DISCUSSIONS

CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN.
Discussion.*

By Messrs. C. E. Gregory, Henry H. Quimby, A. C. Janni, Philip Aylett, and L. J. Mensch.

C. E. Gregory, Assoc. M. Am. Soc. C. E.—The speaker was the Designing Engineer in charge of the design of bridges built by the Board of Water Supply, as referred to in this paper, and wishes to make a few statements to amplify the relation of the work as done by the Board of Water Supply to this paper, and to put others who were concerned in the design of these bridges in a right position in regard to it. The first point he desires to make is this: Mr Gregory.

The authors reach the conclusion that in practically all cases the three-hinged arch is more economical for long spans than the ordinary fixed-end arch. When designing the Traver Hollow Bridge, the speaker did not, and does not now, concur in such a conclusion. The three-hinged type was selected largely because of expected slight settlement in the foundations, rather than the economy of the type of bridge. The preliminary estimates indicated that a three-hinged arch would be more expensive than a fixed-end arch.

The economy of the divided pier is brought out by the authors, but the manner in which the design of the piers of the Rye Outlet Bridge was arrived at differed considerably from that described in the paper, although meeting the assumptions of the authors. The interesting and unusual feature of this bridge is that it is an arcade of five reinforced concrete arches supported on exceptionally high piers. It was found that the yielding of the top of a high pier of ordinary shape under the unbalanced live load arch thrust (the height of the highest pier in this bridge being a little more than 100 ft.) would be sufficient to

* Continued from December, 1913, *Proceedings*.

Mr. Gregory. create considerable stress in the arch without exceeding the usual limits of stress or creating tension in the pier. The design was made to limit the yielding of the piers to that which could be safely and economically provided for in the arches. The entire arcade was treated as a complete elastic structure, and determination was made of what proportion of the unbalanced thrust from a loaded arch would be carried by the adjacent pier and by other arches and piers of the arcade. The shape and size of pier adopted was found to be necessary from calculations made as just outlined.

This work was done under the speaker's supervision by Mr. George L. Bennett and Orrin L. Brodie, M. Am. Soc. C. E., who were at the time assigned to Mr. Smith's staff, but worked on this problem under the speaker's immediate direction.

Mr. Quimby. HENRY H. QUIMBY, M. AM. SOC. C. E.—Experience in the design of concrete arches confirms the opinion expressed by Mr. Gregory regarding the relative economy of fixed and hinged rings, and the justification for the use of hinges. Computations for comparison of designs in several cases have failed to show any saving in cost of a three-hinged type over a fixed type when the ratio of rise to span was fairly good. Hinges are justified only when the foundations are unsatisfactory, or in cases where the arch is very flat. In a flat arch the crown movement due to change in length of ring from temperature is greater than in a high-pitched one, and the temperature stress may thus become great enough to warrant the use of a hinge to avoid it. The arches shown in the paper, however, are not flat, but have a good rise.

The authors give estimated quantities of concrete required for the arch rings in the two designs shown, and represent the fixed type as requiring 90% more concrete than the hinged. This is hard to understand. A fixed ring should require only a little more than a hinged ring. The depth at the crown must be about the same for each because the horizontal thrust is about the same, and any theoretical eccentricity, due to unsymmetrical loading, in the fixed ring will probably be fully matched by the effect of friction of the hinge in the other case, for the inevitable deflection caused by the weight of the floor imposed after the closure of the arch will be likely to produce the effect of eccentric thrust. At the quarter points the hinged ring will need considerably more depth than the fixed ring, and it will be only at the springing line that the fixed ring will be deeper and require more concrete. As the cost of effective hinges is quite considerable, the net result is generally the reverse of that reported in the paper.

If it be said that the greater depth of the fixed ring design in the paper was made because of temperature stress, the reply is that temperature stress necessarily increases with the depth of the ring, and adding to the depth because of such stress is somewhat like piling a

load on a horse's back to enable him to pull a heavier wagon—the poor legs may be overtaxed. Mr. Quimby.

It would seem, therefore, that there must be something wrong either with the estimates or with one of the designs.

The stone hinges shown in the paper give the impression of being likely to develop friction if they work at all, and this will entail eccentricity of stress in the arch ring, to that extent detracting from the only advantage of the hinge.

The matter of appearance of an arch is important in many places, and the bulging at the quarter that is characteristic of the ordinary hinged ring is distinctly ungraceful and displeasing to the miscellaneous eye which cannot understand the reason for it, while the eye of the designer who knows that the point of greatest shear is the point of greatest single stress, desires to see the arch thickest at the spring where that stress is the greatest.

One very important consideration in a discussion of stress in a masonry arch is the method of construction, and it is generally far more vital than temperature fluctuations. The old-fashioned method of laying up an arch ring from the springing line to the crown, with the attendant progressive deformation of the centering and consequent cracking of the earlier portion of the ring practically reduces the working thickness of the ring to a fraction of that designed, and may be the real cause of some failures that have occurred. The proper way to avoid cracks or initial stresses in the arch ring due to settlement or compression of the centering during construction is to build the ring in voussoir blocks with narrow key spaces between them. Then when all the blocks are made, and the centering has, therefore, nearly its full load, after the blocks have seasoned and shrunk, cast the keys. Of course, if the arch is small enough to permit the casting of the full length of the ring in one day—say within 10 hours—the separation into blocks will not be needed, because the first portion cast will be still plastic enough when the ring is closed to adjust itself to the deformed centering. It is impracticable to eliminate initial stress wholly, for any of the dead load that is added subsequently to keying will cause some deformation of the ring and consequently initial stress somewhere; but this should never be enough to be serious.

The claim advanced in the paper that concrete will weather better than good sandstone, and almost if not quite as well as granite, is hardly to be expected from an experienced observer of concrete work. If the authors know of any formula that will produce concrete with such a desirable quality, they should give it to the Profession. It is a poor quality of building stone that will not hold its surface and its edges better than the best concrete. The statement in the paper sounds a little over-enthusiastic. The friends of concrete will promote its use more effectively by avoiding extravagant claims for it.

Mr. Quimby. Another point in the paper that seems not entirely clear is the claim of marked advantage in the I-shape of arch rib. The arch ring is a column—not a beam. If the curve be true and the line of thrust be central, there is absolutely no beam action. Of course, the width—horizontal thickness—of the rib, and its depth or vertical thickness, should both be theoretically adjusted to make a column that will fail by crushing rather than by buckling, and this will require some minimum width. As regards lateral buckling, any given width is as effective at the mid-depth as at the flanges, and, in any case, suitable transverse bracing should be provided. The method of estimating the saving due to the coring out of the concrete at the sides is hardly fair, because a very large part of the cost of such concrete is in the support of it, and any increase or decrease in quantity will be proper to be charged at only the cost of the material and placing. Besides, the extra cost of the additional form work necessary for the paneling will materially reduce the seeming economy in material.

Replying to the question of a member as to the observed actual range of temperature in concrete arches: Two bridges, constructed within the past 5 years, with electrical resistance thermometers embedded at the mid-line, show temperatures in the concrete ranging far less than is usually assumed. The depth of embedment in one case is 2 ft. 6 in. and in the other, 4 ft. 9 in. The former is a spandrel filled arch and the other is an open spandrel. The maximum range of temperature recorded thus far has been from 31° Fahr. in winter to 73° in summer, and measurements of change in elevation of crown made at the same times show a rise and fall computed to correspond to a change of temperature throughout the arch ring of 45° Fahr. between the extremes of cold and heat.

Mr. Janni. A. C. Janni, M. Am. Soc. C. E.—The statement, among others, made concerning the quickness, simplicity, and certainty of the design for three-hinged arches, deserves careful investigation, and should not be accepted off-hand, or as a matter of course.

Therefore, it is not without interest to recall what both theory and experience teach with reference to the equilibrium of this particular kind of construction, and to ascertain to what extent a designer may trust his results.

If S and S_1 (Fig. 5) are the two contact surfaces of the left hinge of an arch, P being the tangent point, and PN the normal to the tangent, T, it is known that as long as the line of action of a force, F, acting on the hinge, is contained within the angle APD (this angle being twice the angle of friction, ϕ, for the material of which the hinge is made), the system will be in equilibrium, provided, of course, the force, F, is a compressive one with respect to the tangent, PT. Similar remarks may be made regarding the surfaces, S' and S_1', of the hinge at the key.

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Therefore, as long as the two components of a force, P, acting on the system, will keep within the two angles, APD and $BP'D$, the system itself will be in equilibrium, and the points, A and B, are to be regarded as the extreme limits of this equilibrium position.

In other words, the system will be always in equilibrium under the action of any force the line of action of which cuts the area $ACBD$.

The foregoing demonstration is very plain, but its foundation is rather indefinite; in fact, it is all based on the assumption that the angle, ϕ, is a known quantity, and, even if that is true, there are two solutions, according to the theory, that satisfy the conditions of equilibrium and, especially for an arch of large and flat span, give quite different results.

Concerning the friction laws, it can be said that, outside of those values of ϕ reported in all engineers' handbooks, which for every-day purposes are tolerably reliable, those laws are not well known at present.

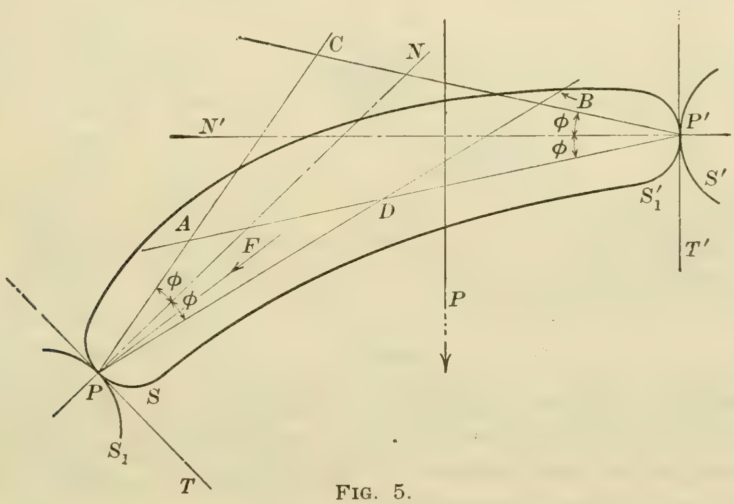

FIG. 5.

The history of the researches on friction is very brief. Amontos, in 1699, started for the first time a research on the friction of some machine of his, without, however, throwing any important light on the general matter.

In 1781, Coulomb took up the researches on this important problem and announced to the world his results, giving out laws on friction, as well as values of ϕ for different materials.

In 1785, Vince started his researches on the same question and rejected, as unreliable, Coulomb's laws and values of ϕ.

In 1829, G. Rennie, who made tests under about the same conditions as Coulomb, found more or less similar results.

Finally, in 1831, Morin presented before the "Académie des Sciences", in Paris, the results on his friction tests, and, though he

Mr. Janni. confirmed Coulomb's fundamental laws, he found numerical values for ϕ which were very different. It is worthy of note that, though Morin made his tests on a larger scale than Coulomb, he did not go farther than a pressure of about 5.2 lb. per sq. in.

As far as the speaker knows, these are the only works on the subject which stand high on account of the illustrious names of their authors; unfortunately, they do not agree among themselves.

Modern investigations, however, without adding anything definite, have found that the friction law, as given by Morin,

$$R = fN$$

where: R = Resistance due to friction;
N = Normal component of the force acting on the body; and
f = A numerical coefficient depending on the materials in contact;

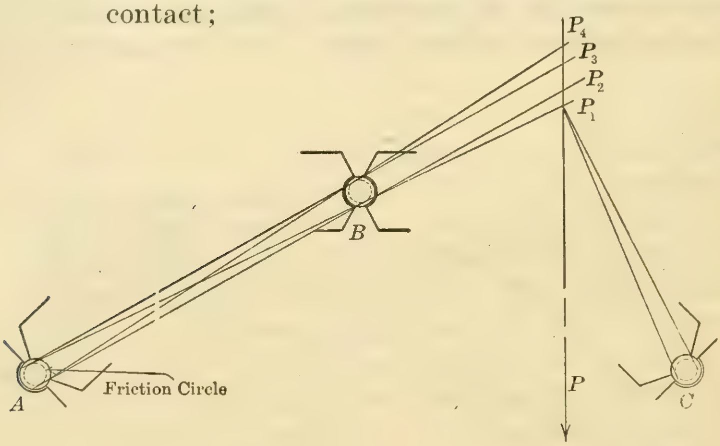

FIG. 6.

is not complete; because, though f is dependent on the materials in contact, it is dependent also on the speed with which one surface moves on the other and on the intensity of the acting force; these laws, however, have not yet been determined.

The only thing known at present is that the value of ϕ, at the start, is between 1.5 and 3 times its value during the movement, and this is quite a wide range when one considers the certainty of results.

Usually, the hinges are metallic, or, even in stone, they have a shape which might be regarded as a hinge; the problem then is solved, as is well known, in the following manner:

Let A (Fig. 6) be the left hinge of an arch; drawing the friction circle with the radius r sin. ϕ (where r is the radius of the hinge and ϕ the angle of friction), it is known that all forces having lines of action which cut that circle will not move the hinge about its center, and all forces having lines of action tangent to that circle will be on the limit position of equilibrium, provided each of these forces has a tendency to compress the hinge against its groove.

If B is the hinge at the crown, it is known that the four common tangents to the two friction circles are limit positions of equilibrium, therefore, according to graphic statics, the problem already admits of four solutions. Mr. Janni.

Taking now into consideration the third hinge, C, if P is a force acting on the system, and P_1, P_2, P_3, and P_4, are the points where the line of action of P cuts the four tangents, from each of those points can be drawn two tangents to the friction at C, which, according to the theory, will resolve the problem.

For the sake of brevity the hinge at B has been regarded as of the same kind as those at A and C.

Apart from the several solutions given by the theory on the equilibrium of this kind of construction, here, too, the indeterminations, resulting from friction, must be considered.

The friction circles can be drawn quite accurately for machinery, or when it is a question of pieces well prepared for laboratory tests, for which test after test has been made so that the whole question is somewhat settled.

Concerning the friction in bridge hinges actually in operation, however, on which the pressure is often so great as to wear off the material at the contract surfaces, it is to be said with regret that there has not been a single test known to the speaker.

It can be seen, therefore, that the quantity, r sin. ϕ, has not, by any means, a known and determined value; in fact, it is rather arbitrary. In other words, the set of four points, P_1, P_2, P_3, and P_4, can be substituted for another set of four points, provided they satisfy the requirements of graphic statics.

Although all the stated indeterminations (and there are others) do not prevent a designer from patching up some kind of design for a three-hinged arch, it is not possible to accept the statement concerning the reliability of such design.

The hinges, however, in a concrete or masonry arch can be used with advantage sometimes during the construction of the arches, when there is no steel reinforcement to take up the tension at the springing line and at the key, due to a possible excessive settlement of the centering; then, after the centering has been removed, the hinges should be concreted. Thus the arch behaves as a three-hinged arch, as far as the dead load is concerned, and as a rigid arch for the live load and temperature. In that instance the hinges, free from the enormous pressure of the dead load, can easily accompany any movement of it, being concreted afterward when their behavior begins to be extremely doubtful.

The granite railroad bridge at Morbegno, Italy, with 229.65 ft. span and 32.8 ft. rise, has been built in accordance with the foregoing method.

Mr. Janni. In comparing concrete and stone bridges, the authors rightly state that one of the advantages of the former is that it is stronger and much more reliable, because of the absence of joints; therefore it is not clear why the paper is a special plea for the three-hinged arch, which has three joints, as compared with the rigid arch, which has none.

Concerning the statement that the trial design for a fixed-end arch takes several days for each analysis, it can be said that it takes no longer than the trial design for a three-hinged arch, and possibly less.

The stresses in the arch with fixed ends, due to dead load and temperature, can be calculated in a day's work easily.

A new method of designing an arch, based on the rather recent theory of the "Ellipse of Elasticity", is described in a paper by the speaker.* This is a purely graphical method and holds, irrespective of the geometrical form of the axis of the arch, irrespective also of the law of variation of the cross-section of the latter, and of the assumptions of loading. In other words, it is possible, by that method, to draw the lines of influence for moments, for any section of the arch, before the loads are applied on the arch, and, as a consequence, the most prejudicial hypothesis of loading for the chosen section is given at once, without any trial.

In that paper, in addition to the theoretical part, the speaker showed a practical application to an arch which he had designed and built by that method.

In contrast with the unreliability of the results in a design for a three-hinged arch, it is not without interest to relate that for a fixed-end reinforced concrete arch, built in France, having a span of 318 ft., the difference between the stresses calculated and the actual stresses reached a maximum of 14.7 per cent.

Every engineer fully appreciates the importance of these results, especially when he considers that, in steel bridges of the best design, that difference reaches 25%, and, when designed on the assumption that there is no friction at the joints, the difference may be more than 100 per cent.

The horizontal thrust, due to temperature changes, is directly proportional to the coefficient of expansion, to the change of temperature in degrees, to the modulus of elasticity of the material, and to the span (measured at the spring points of the geometrical axis of the arch); but it is inversely proportional to the inertia moment of the whole elastic arch with respect to the horizontal (if the arch has its spring points at the same level) passing through a certain point called the center of gravity of the elastic arch.

If, for the same span, arches with different rises are drawn, it can be seen that, although the factors directly proportional to the hori-

* *Journal*, Western Society of Engineers, May, 1913.

zontal thrust remain unchanged (the variation of the span of the geometrical axis may be disregarded), the moment of inertia of the arch increases rapidly with the increase of the rise of the arch; so that the effects of a change in temperature diminish in importance with the increase of the rise. Mr. Janni.

The shrinkage of the concrete due to setting and that due to the dead load are two important causes of stress to be taken care of, especially the former. If the arch is very flat and has a large span, the best and most economical way to take care of the shrinkage due to setting is by neutralizing its effects, as much as possible. A span of 170 ft. with 17 ft. rise, and two spans of 140 ft. with 14 ft. rise, designed by the speaker, have been built, breaking the arches into voussoirs each about 20 ft. long, and leaving between each two consecutive voussoirs a small voussoir, or key, of 2 ft. The voussoirs were cast first and the keys were not cast until the last concrete of the voussoirs was 10 days old, thus avoiding, for the 170-ft. span, for instance, important tensions on the intrados at the key and on the extrados at the springing line, especially during the winter, due to a shortening of $\frac{7}{8}$-in. of the geometrical axis of the arch, which otherwise would have taken place.

If, as the authors state, an average pressure of 12 000 lb. per sq. in. on the crown hinge of that arch was calculated, it must be inferred that the maximum pressure on the hinge was about 24 000 lb. per sq. in. at the theoretical point of contact with its groove, and it may be assumed that it was reduced to zero at each end of the groove.

The question then arises: At such a vital point of a bridge may the material be submitted to work at a rate which is never allowed for such material at other much less important points; and that without discussing the practical inadvisability of trusting on a $2\frac{1}{2}$-in. space the stability of such a comparatively enormous mass of material?

In dealing with such pressures, however, there is no longer any question of the possibility of assuming that at that point there is a hinge, ready to work; it is no more a matter of design, but of guesswork.

When a hinge of such small diameter is placed at the key, a malicious attempt to wreck the span could be successfully carried out.

Another kind of hinge—fortunately, very seldom used—is a ball-and-socket joint of cast steel consisting of a hemispherical protuberance fitting into a hemispherical cavity. Any horizontal displacement of the crown of the arch, for a structure of this kind, cannot be contemplated seriously; there is no reason for using such a hinge. In the construction of a three-hinged arch, the use of a ball-and-socket hinge means simply the addition of more friction at that joint.

Having an arch with only two ribs, and that arch hinged with the ball-and-socket type, then, at each joint cross-section there will be

Mr. Janni. at least two hinges. Admitting, for the moment, that a horizontal displacement of the crown might take place, it will be admitted also, that it will be very small.

On account of the rigidity of connection between the two ribs, however, there is no possibility of a horizontal rotation of both hinges, even disregarding friction and admitting, of course, the elastic deformation of the whole system; but the effect of that displacement will be only to decrease the pressure on one hinge at the expense of the other.

However, as the use of a hinge of that kind shows that the designer assumed the possibility of a really tangible horizontal displacement of the key and the consequent horizontal rotation of the hinge, it is not without interest to ascertain what would happen to the arch if a horizontal rotation took place at the key, no matter how small it might be, provided, however, that it be a tangible quantity.

Assume that the hinge has a horizontal rotation of only 10′, and that the two hinges are 30 ft. apart; then, while one hinge presses in its socket during a rotation of 10′ (always disregarding friction), the other would be lifted out of its socket a distance of about 1 in. Although this will not happen, on account of the elastic deformation of the system, it is fair to admit that, at that joint, the horizontal thrust is zero (without speaking of the pressure concentrated on the rotating hinge).

What happens at the key, according to the designer's assumption, will happen also at the springing line, but the order will be reversed. Then the question arises: Did the designer calculate the stresses in his elastic system under such abnormal conditions of equilibrium? If he did, it is certain that he will never advocate the ease, speed, and certainty of a design for a three-hinged arch, especially with ball-and-socket joints; if he did not, then his enthusiasm for structures of this design is quite natural.

M. Mesnager, wisely concerned about the secondary stresses arising from the use of the hinges in arch bridges, and for which there is no way, at present, to determine their exact amount, thought that, a half-rigid joint "semi-articulation", the working behavior of which could be easily determined, would be preferable to the usual hinges.

For that purpose, several years ago, under his personal supervision, the Testing Laboratory of the "École des Ponts et Chaussées" started researches on a joint, Fig. 7, built in reinforced concrete according to his plans. Without going into details concerning his tests, it is enough to repeat one of his remarks:

"Le moment qui a été nécessaire pour fair fléchir la semi-articulation quand elle était noyée dans le béton, n'était pas négligeable."

Then he concluded by saying that it was possible, perhaps, to free the joint, *J*, from concrete and fill the remaining space with some kind of non-acid asphalt.

Mr. Janni.

Evidently, a hinge of this kind, if developed properly, could not be expensive, and, at the same time, would eliminate the secondary stresses depending on friction, at least.

Concerning the saving of material by building the ribs of I-shaped section, it may be remarked that such a saving, in a general way, does not mean, necessarily, a saving of money. It would be rather absurd to think that a marble statue costs less than a block of the same material having the same external dimensions, only because the actual volume in the statue is less than that in the block. In addition, however, to all doubts expressed by other speakers concerning the cheapness of an I-shaped rib, as compared with the rectangular rib, another remark may be made: The three-hinged arch, on account of its lack of rigidity, requires a stronger system of stiffeners than that needed by a fixed-end arch, and the quantity of concrete in the stiffeners for the former must, necessarily, be greater than in those for the latter; therefore the statement advanced by the authors, that the quantity of concrete was the same in each case, cannot be accepted.

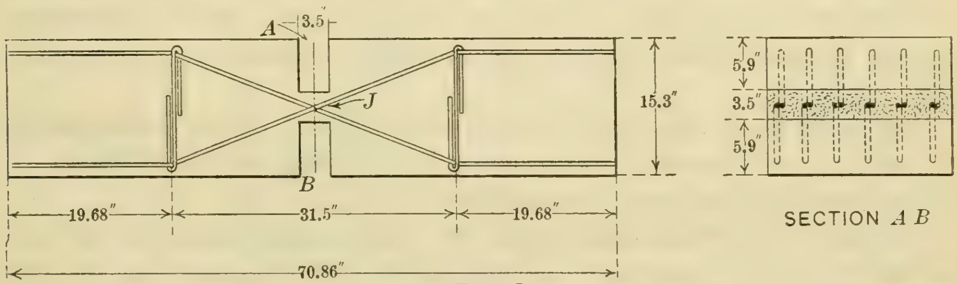

FIG. 7

In conclusion, it is to be noted that the statement concerning the complication of design for a fixed-end arch and the ease with which such an arch can be upset by a slight movement of one of its supports, is not altogether justified. This complication of design, so much complained of, is justified only if the designer expects to calculate the arch as simply as he can pick out, from the "Carnegie Handbook," the I-beams for a floor.

Concerning the latter remark (which was correct several years ago) it can be said that, by the theory of "virtual work" or, more elegantly and speedily, by the method of the "ellipse of elasticity", the designer can easily take care of such dreaded displacement of support, and even of a yielding of the soil on which the footings rest.

There is no doubt that the design for an arch is one of the most delicate problems of engineering; but, after all, it may be done without any great trouble if the designer is sufficiently familiar with his work.

The theories used are not monopolized by a few persons, but are quite accessible to any one of training and application. Fortunately for the designer, those theories cannot be reduced to "rule-of-thumb",

Mr. Janni.

for several reasons. If an engineer, satisfied with what he was taught at school, does not keep in touch with the daily progress of science, he runs the risk, nowadays, of being behind the times. If, for instance, in designing an arch, he still assumes first that the arch is entirely loaded and then, that it is half loaded and half unloaded—still believing that, in this way, he ascertains the maximum stresses in that arch —he is flatly wrong; those assumptions were made several years ago, because there was then no means of ascertaining, conveniently, the most prejudicial hypothesis of loading; at present, however, that method of computing stresses is antiquated as well as inadequate.

Mr. Aylett.

Philip Aylett, Assoc. M. Am. Soc. C. E. (by letter).—In America, the ribbed arch, especially the three-hinged type, has not received that recognition of its merits which it deserves. Its economy and advantages are better known and more highly appreciated in Europe, where it has been used extensively under almost all conditions of site and traffic, and in great variety of form and design.

Comparatively few concrete bridges of the rib type proper (hinged or fixed) have been built, as yet, in America, and these only within the past decade. The literature on the subject, therefore, is comparatively scant.

The authors of this paper deserve the thanks of all interested in the use and development of this type in America, where its future is so promising and its field so extensive.

The use of three-hinged arches on compressible material is fully justified by ample experience and precedents. The foundations of abutments and piers in such cases, however, demand more careful investigation, study, and design. A fixed or a two-hinged arch may also often be built with entire safety on compressible material, provided the foundations are properly designed and built, including the use of batter piles and other methods of taking up thrust. This also is justified by numerous structures under such conditions now extant, both in Europe and America.

The seeming economy of the rib of I-section, as compared with the rectangular, will not show so vividly in practice as on paper. This saving in concrete in the I-section will be largely, if not entirely, offset by the greater expense caused by the additional material and labor involved in making and placing the I-forms and depositing the concrete therein. Forms for such sections also necessitate the cutting up of the lumber into many short lengths, thus making much of the material unfit for other use. For instance, Fig. 8 shows an I-section form designed for the purpose of economizing in material and in the labor of making, placing, and dismantling it. This form is also capable of repeated use, as it is built in sections with this in view. Yet the

additional material and labor involved, in comparison with forms of rectangular section, is apparent at first glance. Mr. Aylett.

In addition, the I-section rib invites and makes possible injury and defects to the permanent structure from which the rectangular section is exempt. No one can foresee or forestall settlements in falsework or centering from timber shrinkage, "taking up", "biting", col-

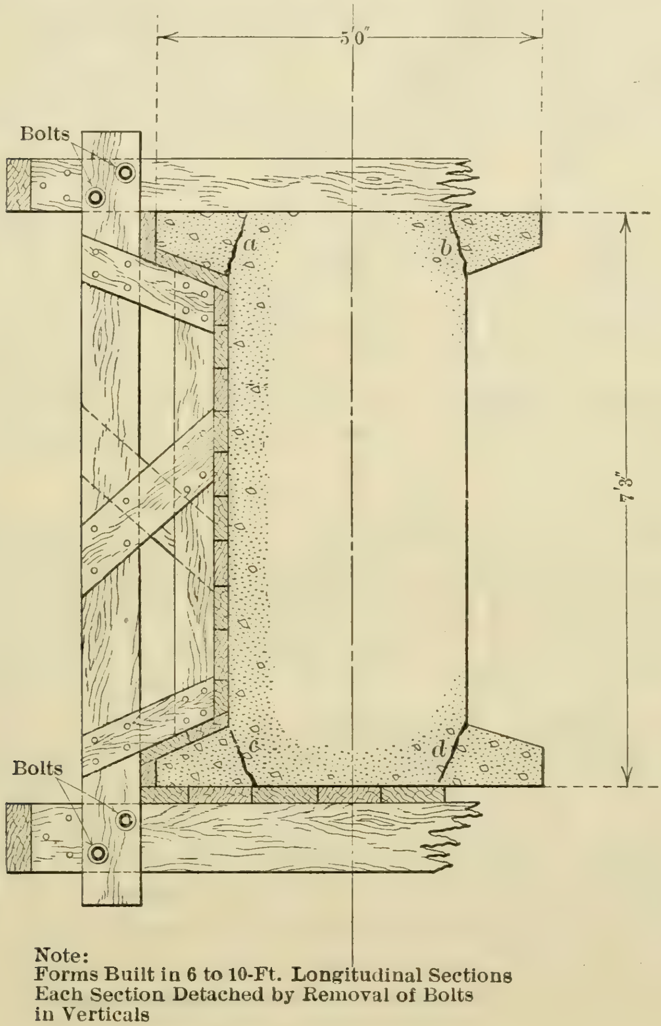

FIG. 8.

umnar shortening, foundation movements, expansion, contraction, etc., etc. Falsework or centering moves upward as well as downward on account of the unsymmetrical placing of the load, or in cases where the falsework becomes wet by rain or flood after being thoroughly dried out. A rectangular arch rib is capable of sustaining more settlement or upward movement of the falsework without injury than is possible with an I-section, because of the projecting flanges of the

Mr. Aylett. latter. Settlements or movements of the falsework prior to the removal of the I-forms seriously endanger these concrete flanges, and will probably cause rupture at or near their junction with the body or web of the section, as at *a, b, c,* and *d,* of Fig. 8.

From a number of cases, it has been observed that the maximum movements of falsework (both up and down) occur during or immediately after the construction of the arch ribs (except in cases of accidents, etc.); because this is the period of the initial adaptation or accommodation of the falsework to its superimposed load. During this period, therefore, the I-forms are always found in position and cannot be dispensed with. The danger of flange rupture is most imminent during the initial stages of arch action; that is, when arch action begins to assert itself, the concrete being comparatively green.

Flange rupture may occur, however, after the rib has passed into the self-supporting stage, or even before arch action asserts itself at all. For instance, if a downward movement of the falsework occurs just as arch action begins to take place, the rib offers some resistance to the movement, though it may be incapable of entire self-support. The result is rupture in the bottom flanges and cracks opening upward, as at *d,* Fig. 8.

If an upward movement of the falsework occurs (on account of swelling, distortion, or accumulation of drift and débris against the forms), flange rupture takes place in the opposite direction, the cracks opening downward, and both top and bottom flanges may be affected, as at *a, b,* and *c,* Fig. 8. Flange rupture prior to arch action is most likely to occur in connection with the use of heavy reinforcing members, such as I-beams, lattice girders, etc., as these, by their own arch action, are capable of resisting the movements of the falsework.

It must also be remembered that at or near the points, *c* and *d,* in the I-section, the poorest and weakest concrete will undoubtedly be found. It is here that the greatest difficulty will be met in securing good concrete, on account of the re-entrant angles in the forms, and the proximity of the reinforcing members thereto. Hence the flanges which must resist the greater stress from such movements are less capable of doing so, on account of the quantity and quality of the material.

The likelihood of flange rupture (especially after arch action in the rib) may be decreased, more or less, by avoiding a rigid connection between the sides of the I-forms and the supporting timbers below. Thus, in Fig. 8, such a precaution may be provided by simply omitting (on both sides) the lower bolts shown in the verticals, and using stop-blocks instead. Slip-joints thus formed would permit the downward movement of the falsework without injury to the flanges, provided the arch had attained the stage of self-support.

Although the authors affirm that "the shape of the piers" is a "most important point in the design", they appear to overlook this and other important points relating to the design and construction of bridge piers, especially in flowing streams. Mr. Aylett.

In the comparative estimates of the braced and solid types of piers, apparently, only the difference in concrete has been considered. The greater length of coffer-dam required by a pier of the braced type (with or without diaphragm or foundation offset) should be considered. In constructing coffer-dams in flowing streams, the important thing is usually time, for more risk is attached to coffer-dam building in flowing streams than in any other operation connected with ordinary bridge building, especially where there is "ice or logs of wood moving rapidly".

The many-sided dam of the braced type pier requires much maneuvering of the pile-driver in order to effect the frequent changes of direction. This involves change of anchorage, backing, going forward, etc., etc., all by men who are not sailors, and while breasting a current, and with risk of disaster by flood.

The solid rectangular pier requires less coffer-dam, less maneuvering of the pile-driver, less exposure to flood at the most critical time, and, consequently, much less time and expense. These are among the important points which should be considered in pier design and construction, and cannot be overlooked in any estimates.

There are, however, other features of the authors' braced pier which appear so serious as to preclude entirely this type from consideration in streams of the character mentioned.

The authors state:

"This type [braced] of pier is especially economical for a bridge in running water in which there is likely to be heavy blocks of ice or logs of wood moving rapidly."

Even prior to 1913 streams of this character throughout the country indicated, by ample evidence and signs, the imprudence of obstructing their flow. Railway officials and engineers, having interest in many streams over extended territory, were among the first to be impressed. Frequent and alarming flood records in numerous streams were observed. Bridge piers which had given many years of service, without trouble or expense, became the causes of claims and litigation on account of damage to adjoining lands by soil stripping and deposits of mud, silt, and débris, resulting from stream obstruction and current deflection.

It was observed that, usually, mud, silt, etc., were deposited in greatest abundance where the water first left the banks of running streams. This occurred in most cases immediately below bridge piers or other current deflectors.

Mr. Aylett.

Railway men and others who have borne the responsibility (night and day) of traffic and maintenance of way during flood periods, know the expense and risk attached to ordinary solid piers in running streams carrying ice and drift. The braced pier recommended by the authors for such locations would prove a veritable drift-trap and, viewing the matter broadly, appears to be a most obstructive and expensive type. Cut-waters, though valuable safeguards, especially against ice, do not prevent the lodgment of all logs and drift, even when solid piers of comparatively less widths are used.

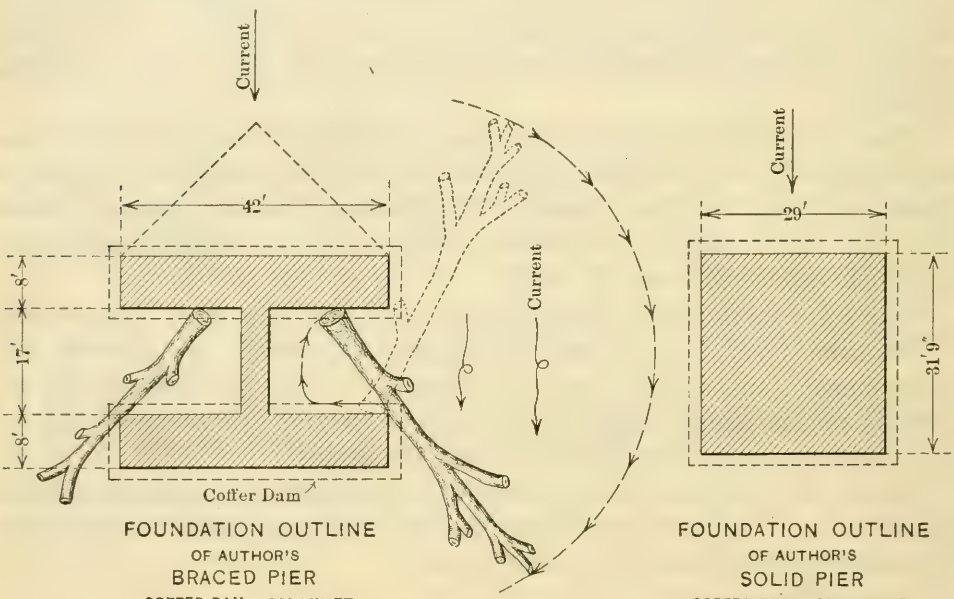

FIG. 9.

The authors' braced type of pier is exposed, not only to the accumulation of drift, etc., on its nose or up-stream end, as in solid piers, but will also catch and hold rigidly logs and drift in the space between the legs on both sides (Fig. 9). Drift thus trapped and held would not float and rise with the flood as it does with solid piers, and as it would be quickly submerged when caught, it would be almost impossible for an ordinary section gang, even with machinery, to dislodge it.

The buoyancy and enormous leverage of a mass of logs and drift, thus trapped, would exert a simultaneous lifting and wrenching action on the pier, increasing with the pressure of the rising flood on the growing mass held firmly before it.

Drift which collects at the noses of solid piers can always be removed by ice-hooks or pike-poles and floated away. Masses of drift cause deep scour around piers, and this would probably extend below the diaphragm, if its depth were only "down to where they [the pier-legs] enter the ground." If this should occur, heavy drift from both

sides might find its way under the diaphragm as well as the braces. It is difficult to reconcile the obstructive braced pier recommended by the authors with the nation-wide conservation movement of to-day, especially in the face of the experience of the West and Middle West in 1913, with its lessons so clearly, impressively, and profoundly taught. Mr. Aylett.

Amid the losses of lives and property, the bridge engineer's lesson stood out above all others—unmistakable and specific:

It was: Conservancy of Rivers by

(*a*) Avoiding unduly wide and obstructive piers;

(*b*) Reduction in number of piers by increase of span lengths;

(*c*) Increasing floor-level elevations.

These would facilitate the discharge and consequently help to lower the flood level. By so doing, other lives and property may be saved, and the sacrifices in the West and Middle West will not have been in vain.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—The statement of the authors that three-hinged arches contain only about half as much masonry as properly designed fixed arches is only true for bridges of very small rise, and the writer would be interested to see the calculations for the bridges shown in Fig. 3, as his own tables do not show such a difference. The fact is that the thrust in both types is practically the same. The bending moment caused by a concentrated load in a three-hinged arch is greatest when the load is 0.211 l from the abutment, and its value is about $P \times \frac{l}{10}$; in a fixed arch a concentrated load produces the greatest bending moment when placed 0.3 l from the abutment, and its value is about $P \times \frac{l}{16}$. Mr. Mensch.

It is customary to calculate a bridge for a uniform load, w, placed at its half span. The greatest bending produced in a three-hinged arch occurs at the quarter span, and amounts to $\frac{w \times l^2}{64}$; in a fixed arch it occurs at the point 0.3 l from the abutment, and amounts to $\frac{w\ l^2}{120}$; at the same time the moment at the abutment is $\frac{w\ l^2}{64}$.

There is nothing to justify the statement that there is a considerable saving in materials in a three-hinged arch over a fixed arch, unless the temperature stresses and the stresses due to the shortening of the arch and the shrinkage of the concrete play an important rôle, which is only the case in arches of comparatively small rise and large depth. Professor Howe, in his treatise on arches, showed that the thrust from a change of temperature in a parabolic arch of constant $\frac{d\ s}{I}$ may be found by the formula, $\frac{45}{4}\ \frac{E\ I\ c\ t}{f^2}$, when E = the modulus

Mr. Mensch. of elasticity, I = the moment of inertia of the crown section, c = coefficient of expansion, t = change of temperature, in degrees Fahrenheit, and f = rise of arch; and that its point of application should be assumed at $\frac{1}{3} f$ below the crown. The writer finds that in a parabolic arch, as usually designed, the thickness of the arch increasing at the abutment to about $1\frac{1}{2}$ to 2 times the crown thickness, the thrust may be found by the formula, $24 \frac{E\,I\,c\,t}{f^2}$, and its point of application may be assumed to be $\frac{7}{30} f$ below the crown. Let E = 2 000 000, $t = 50°$, $c = 0.0000055$, or $E\,c\,t = 550$.

From a change of temperature, the bending moment is found to be,

at the crown, $\frac{24\,I}{f^2} \times 550 \times \frac{7}{30} f = 3\,080 \frac{I}{f}$;

at the abutment, $\frac{24\,I}{f^2} \times 550 \times \frac{23}{30} f = 10\,120 \frac{I}{f}$.

For a width of 1 ft. of the arch ring, $I = \frac{12\,d^3}{12}$, when d = the thickness at the crown; the moment at the crown $= 3\,080 \frac{d^3}{f}$, and the moment at the abutment $= 10\,120 \frac{d^3}{f}$.

If we consider that the moment of resistance of the crown section for a width of 12 in. (neglecting the reinforcement) $= \frac{12 \times d^2}{6} = 2\,d^2$, and that the moment of resistance at the abutment, when the thickness is twice the crown section $= \frac{12 \times 4d^2}{6} = 8d^2$, we can obtain for the stresses in the extreme fibers caused by the change of temperature, by dividing the moments by the section modulus, the simple expressions, $1\,540 \frac{d}{f}$ for the crown section, and $1\,265 \frac{d}{f}$ at the abutment, whatever the span of the arch.

The writer will endeavor to compare the two bridges shown in Fig. 3, as far as the few measurements given will allow. He will assume that the thrust per linear foot may be found by the formula, $\frac{850 \times 128.5^2}{8 \times 25} = 70\,000$ lb. The live load of 100 lb. per sq. ft. distributed over a width of 29 ft. will amount to $\frac{100 \times 29}{17} = 170$ lb. if

distributed over the two arch rings 17 ft. wide; and the bending moment in a three-hinged arch at the quarter span Mr. Mensch.

$$= \frac{170 \times 128.5^2}{64} = 44\ 000 \text{ ft-lb. per lin. ft.}$$

The crown thickness of the arch is 22 in., and assuming that the thickness at the quarter point is 33 in., the stresses at the quarter point may be found as follows:

The compressive stress $= \dfrac{70\ 000}{33 \times 12} = 177$ lb. per sq. in.

The stresses in the extreme fibers from the bending moment

$$= \frac{44\ 000 \times 12}{12\,\dfrac{33 \times 33}{6}} = 242 \text{ lb.}$$

or a maximum compressive stress of $177 + 242 = 419$ lb. and a maximum tension of $242 - 177 = 65$ lb.

The average thickness of the arch is $22 + \frac{2}{3} \times 11 = 29.33$ in.

In a fixed arch it will be necessary to assume a greater thickness of the arch at the crown, on account of the temperature stresses; therefore, the thickness at the crown will be assumed to be 28 in. and that at the abutment, 56 in. The maximum bending moment caused by a uniform load, w, per square foot at the crown is about $\dfrac{w \times l^2}{320}$, and the compressive stress from the thrust $= \dfrac{70\ 000}{12 \times 28} = 208$ lb. per sq. in.; the stresses in the extreme fibers, from the uniform load, $= \dfrac{170 \times 128.5^2}{320}$, divided by $\dfrac{12 \times 28^2}{6} = 56$ lb. per sq. in.

As shown before, the temperature stress in the extreme fibers $= 1\ 540\,\dfrac{d}{f} = 144$ lb. per sq. in., or a maximum compressive stress of $208 + 56 + 144 = 408$ lb. and a minimum compressive stress of $208 - 200 = 8$ lb. per sq. in. At the abutment, the thrust is $70\ 000 \times 1.28 = 89\ 500$ lb. and the compressive stress $= \dfrac{89\ 500}{12 \times 56} = 133$ lb. The maximum bending moment from a uniform load

$$= \frac{170 \times 128.5^2}{64} = 44\ 000 \text{ ft-lb.}$$

The section modulus at the abutment $= \dfrac{12 \times 56^2}{6} = 6\ 280$ in.^{3}, and the stresses in the extreme fibers $= \dfrac{44\ 000 \times 12}{6\ 280} = 84$ lb.

Mr. Mensch.

The temperature stresses $= 1\,265\,\frac{d}{f} = 118$ lb., or the maximum compressive stress $= 133 + 84 + 118 = 335$ lb. per sq. in., and the maximum tensile stress $= 202 - 133 = 69$ lb. per sq. in. The average thickness of the arch $= 28 + \left(\frac{1}{3} \times 28\right) = 37.33$ in.

The stresses from the shortening of the arches will modify the results only very slightly in this case. Although both bridges have about the same stresses, the average quantity of concrete is only 27% greater in the fixed than in the three-hinged arch. Or, assuming that 200 cu. yd. for the three-hinged arch is correct, the saving amounts to only 54 cu. yd., or $675, which is more than counterbalanced by the cost of steel hinges.

The authors state that the slightest movement of the abutments will develop cracks in the arches. This is not the case. A temperature drop of 50° is equivalent to a movement of both abutments of $\frac{1}{3\,640}$ of the span, or, in this case, 0.42 in., and it is a poorly designed abutment in which this movement will be seen. Such a movement would increase the stresses at the crown to $408 + 144 = 552$ lb. in compression and $144 - 8 = 136$ lb. in tension, and at the abutment to $335 + 118 = 453$ lb. in compression and $69 + 118 = 187$ lb. in tension. It is known that ordinary slabs do not crack under stresses less than 300 lb. per sq. in.; hence the factor of safety against the first crack is still ample.

It is true, however, that the temperature stresses and those due to the shortening of the arch would be three or four times as great in the same arch with a rise of only one-tenth of the span, and the advantage of the three-hinged arch in such a case is unquestioned.

In a three-hinged arch, great stresses may be caused by the unequal settlement of the staging, and it has often happened that the hinges have been thrown out of line and out of center, and have had to be dug out and replaced. The failure of a large bridge was due to this cause. This is the main reason that fixed arches are preferred by experienced engineers to the three-hinged type.

It must also be mentioned that three-hinged arches settle considerably more than fixed arches after the centering is struck, and also cause more vibration under moving loads.

In three-hinged arches, the use of deep ribs is a decided saving, which is not the case in all fixed arches. The writer has advocated* the use of I-sections for spans of more than 300 ft., and for spans of from 200 to 300 ft. he advocates T-sections.

The rule given by the authors for finding the center line for a three-hinged arch can be simplified by assuming the entire arch to be loaded by one-half of the uniform load per square foot. This, according to the elastic theory, gives the most favorable line of pressure.

* "The Reinforced Concrete Pocket Book."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE EFFECT OF SATURATION ON THE STRENGTH OF CONCRETE.

Discussion.*

By Henry H. Quimby, M. Am. Soc. C. E.

Henry H. Quimby, M. Am. Soc. C. E.—Mr. Worcester's discussion brings up recollections of some early experience with concrete. Certain 6-in. test cubes made by an inspector in the field yielded very unsatisfactory results when crushed, breaking as low as 500 lb. per sq. in. The cement, which was a standard Portland brand, and the sand, which was New Jersey bank sand, were suspected and retested, but were found to be normal. The concrete was friable—the edges easily rubbed off with the thumb—and it was very absorbent. Tensile test briquettes of the mortar—1 to 3—made at the same time as the cubes, tested as low as 39 lb. at 7 days. Some broken pieces partly immersed in water to observe their absorption were noticed after a few days to have considerably higher tenacity. Similar experiences on other work later, and also some experiments, made it clear that premature setting and drying out prevents concrete from properly ripening. Mr. Quimby.

The trouble appears in hot weather when all the materials have been heated by exposure to the sun, and perhaps when, in addition, a quick-setting cement is used. When concrete is found to be deficient in strength because of premature drying out, continued saturation with water will increase its strength, but will probably not raise it to the normal point unless the saturation be continued for a very long time.

It is evident that concrete wants its moisture and time together.

In order to secure uniform conditions of treatment of test cubes, the rule has been adopted for all inspectors to bury each cube in moist earth as soon as it is removed from the mould, and keep it there until it is sent in to the testing laboratory to be crushed.

* Continued from December, 1913, *Proceedings*.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

BY MESSRS. L. M. WINSOR, H. W. KING, AND ROBERT E. HORTON.

L. M. WINSOR, ESQ.† (by letter).—One of the most crying needs to-day of the entire irrigated West is careful attention to accurate methods of measuring the flow of streams and to the proper distribution of this flow. Through the continual development of larger areas under irrigation, water has become or is rapidly becoming much more valuable than the land it covers. Water users are no longer satisfied with the loose methods of division which were put into practice during the early years of irrigation development. Each season brings new demands for an accurate system of measurement and distribution. For this reason it is high time that the Engineering Profession perfect some accurate, economical, and at the same time simple, device to meet this demand. Mr. Winsor.

The weir in its various forms has been accepted as the device best suited to the needs of the irrigator. Before adopting it as a standard, however, it would seem advisable to make a more careful study of its use, under actual field conditions, than has ever been made. Most of the observations thus far have been conducted under conditions quite foreign to those which are met in the operation of a canal system. The early experimenters used, in the main, larger streams than the ordinary irrigation stream requiring measurements, and the recent experiments were conducted under laboratory conditions which are seldom applicable to the ordinary irrigation stream.

* Continued from December, 1913, *Proceedings*.

† Asst. Irrig. Engr., Office of Experiment Stations, U. S. Dept. of Agriculture, Logan, Utah.

Mr. Winsor. The work being done at the new laboratory at Fort Collins, Colo., under the immediate direction of Professor C. M. Cone, of the Division of Irrigation Investigations, U. S. Department of Agriculture, will without question throw a great deal of new light on the matter of water measurement, and is no doubt the most important work of its kind going on to-day. This laboratory is equipped to make measurements under practical conditions, and the results there obtained will be of untold value to the irrigators of the West, and to the irrigation engineer.

The sharp-crested rectangular weir without end contractions, recommended in the paper, is without doubt one of the best forms, if not the best, now in use, where it can be put in place properly. It is not, however, as easy to erect for ordinary use as is the weir with end contractions. The latter can be placed directly across the channel without the trouble or expense of constructing a channel of approach, the only requisite being the enlargement of the up-stream portion of the stream bed to a sufficient area. In practice, the weir, or any other device, must be adopted by degrees. Farmers or canal operators object at first to any device which is expensive; therefore it is well to give due consideration to one which will give results approaching accuracy, and at the same time admit of construction. When canal operators are educated to the proper point, the most accurate of the various devices may be recommended, even if expensive.

In any case, if the suppressed weir is to be used, it is very important that the area of the up-stream channel be enlarged considerably in order to reduce the velocity above the approach to the weir to such an extent that it will not affect the flow.

The point made by the author that the velocity over a suppressed weir is greater than that over a contracted one is very important when applied to most western streams, which carry a great deal of sediment. This deposit of sediment above the weir is one of the main factors in prohibiting its use under irrigation conditions.

In the writer's opinion, the discussion of the various forms of broad-crested weirs, as applied to the present needs of the country, is less important than the portion devoted to the sharp-crested weir, and particularly to the first three plates or diagrams. Great credit is due the author for his originality in working out this system, and particularly for his clear and concise comparisons between the various experiments.

Mr. King. H. W. King, M. Am. Soc. C. E. (by letter).—The weir experiments of Francis, Fteley and Stearns, and Bazin have been examined so minutely, and have been grouped and studied so carefully and thoroughly by so many able hydraulicians that it would hardly seem possible to evolve anything really new from them. The author, however, by his method of combining into working diagrams the results

of these experiments and the more recent ones of the Cornell and University of Utah Hydraulic Laboratories, has done much to simplify the use of this mass of data. The contribution is a most valuable one, because of the new methods of analysis developed, as well as the practical value of the results obtained. Mr. King.

Inasmuch as the author did not endeavor to derive a general formula of discharge, he was able to obtain several formulas for each series of observations which fit very closely each value given. These exact formulas were in turn used in deriving the quantities used in the diagrams. It is evident that the author is consistent in his methods of constructing these diagrams, and they represent accurately the results obtained from the experiments. There can be no doubt that quantities can be picked off the diagrams with as great accuracy as is justified by the results of the original experiments, and, doubtless, they furnish the most ready and reliable means yet provided for solving problems of the type to which they apply.

This paper, however, should be considered as a very comprehensive study of one phase of the subject of weirs rather than a general analysis. The author takes up the single case of a weir in a channel of rectangular cross-section in which the length of the weir is equal to the width of the channel. This is the condition under which many, if not most, of the weir experiments above referred to were made. In constructing a weir for the measurement of water, it frequently happens that these requirements may be fulfilled. It is not true, however, that this may be done in all cases. In many cases the engineer wishes to measure the flow of water by the weir method in a canal of trapezoidal section or a stream of irregular section. There is, moreover, a question as to whether it will be practicable under all conditions to require the measurement of irrigation waters by suppressed weirs in rectangular channels, as the author suggests. These ideal conditions should be obtained when practicable, but there seems to be no good reason for limiting the use of the weir to this one case.

Occasions will continually arise when it will be desirable to construct weirs across other than rectangular channels, and there will consequently continue to be need for an accurate solution of these problems. It also appears that we are not yet ready to dispense with the weir with end contractions, and, therefore, there is still need for a means of determining the discharge over such weirs. It is doubtless true that, for maximum accuracy, with our present limited data and knowledge regarding the flow over weirs, the channel of rectangular section and suppressed weir should be used where practicable. This, however, is due to the fact that the experimental data thus far obtained apply more particularly to weirs of this type, and not

Mr. King. because those constructed under other conditions may not be possible of as exact solution.

In order that a method of solution of weir problems may be of general application, it must provide for a channel of approach of any cross-section. It should also, with proper modifications, include the weir with end contractions. It would appear, therefore, that the foundation of the method of solution must apply to the weir discharging from the theoretically quiet pond. The coefficient of discharge under such conditions is well known to be very close to a constant quantity within the range of head covered by our present experimental knowledge. The deviation of this coefficient from a constant, therefore, is due almost entirely to effects of velocity of approach. The present weir formulas, with the exception of Bazin's, use this general method of solution; that is, they determine the discharge from a still pond and correct the observed head, so as to include the increased discharge due to velocity of approach. That these formulas do not give results of sufficient accuracy is unquestioned. The trouble, however, does not lie in erroneous theory or assumptions, but in being based on insufficient if not partly unreliable experimental data.

There still remains to the Engineering Profession the problem of providing a satisfactory general method of solution for all weir problems. Such a method of solution must be based largely on new data, which should cover a wider range than those existing, and be of unquestioned accuracy. New experiments are needed to give more definite knowledge of the discharge over weirs without velocity of approach, which will replace the assortment of formulas in use at present. There is even greater need of complete data which will give a clear understanding of the laws governing the flow of water, the distribution of velocities in the channel of approach, and the effect of these laws on the discharge over weirs. At present, adequate facilities for such experiments are not available, but it is to be hoped that a means will soon be provided for obtaining these much needed data. With them a ready means should be provided for solving all weir problems within the required limits of accuracy.

Mr. Horton. ROBERT E. HORTON, M. AM. SOC. C. E. (by letter).—The writer regrets that he has been unable to find time to give this paper the careful perusal it undoubtedly deserves.

Heretofore, a thin-edged weir, constructed substantially in accordance with the rules laid down by the late James B. Francis, Past-President, Am. Soc. C. E., has commonly been designated a "standard weir", and has often been accepted by the Courts as a correct method of measuring water, whether or not it was so in fact. The writer agrees with Mr. Lyman to the extent that a suppressed weir is an equally good "standard", but questions the propriety of legalizing this

or any other form of weir as a "standard". It seems to the writer that there are now a considerable number of forms of weirs, all sufficiently well calibrated to serve as excellent standards in particular cases, no one of which is the best standard for all cases. Mr. Horton.

The hydraulics of weir discharge is by no means as simple a matter as it may appear to a neophyte. A good engineer can get good results with good weirs of various forms. The writer does not hesitate to say that he has seen "Francis Standard" weirs abused—and the results sworn to in Court—in a startling manner. Such abuses are usually coupled with the opinion, tacit at least, that as the weir is denominated "standard", it is consequently fool-proof, and the element of personal equation is eliminated.

Great injury has been done by writers of textbooks on hydraulics by placing undue stress on the merits of so-called standard weirs and orifices, especially the latter, as compared with other forms. Almost any kind of a round-edged orifice, with a reasonable edge radius, is subject to less variation in coefficient in practice, and is less likely to be obstructed or change its form, than a thin-edged orifice. The same is true to some extent for weirs. It is also true in both cases that the round-edged form discharges more water for an opening of a given size then the thin-edged form. This is a matter of some importance, in the case of orifices, in measuring the supply of water-wheels through flume or bulkhead openings where water is sold or leased in square inches or other units. It is a matter of great importance in designing flood spillways for dams, because, with a given length, the round-crested spillway, having a larger coefficient, will discharge a flood with a less depth of back-water than a broad flat or a thin-edged weir. The floods in the spring of 1913 demonstrated pretty clearly that spillways of dams form the best practicable method of measuring the maximum flood discharge of rivers, and for that purpose, ogee or round-crested weir sections are the best practicable standard.

Mr. Lyman describes various base formulas which, with suitable coefficients, have been used for calculating the discharge over weirs of various forms. In the original deduction of the U. S. Geological Survey experiments of 1903*, the writer chose the theoretical formula, which had also been adopted as a base formula by Francis for thin-edged weirs:

$$Q = C\,L\,H^{\frac{3}{2}}$$

The reasons were as follows:

(1) The results in this form for irregular sections are directly comparable with the discharge of a thin-edged weir, when the latter is calculated by the formula then in most common use.

* As described in Water Supply Papers Nos. 150 and 200. The table of multipliers appearing at the end of these papers was inserted after the paper left the writer's hands, and without his sanction.

Mr. Horton.

(2) The coefficients for many forms of weirs of irregular section are not continuous functions of the head, so that the full history of the discharge under a wide range of heads cannot be represented by any single formula with a constant coefficient.*

(3) If, as is true in many, but by no means all, cases, a variable coefficient must be used in formulas for such weirs, then it seems better to throw all the variation into the coefficient and make the computation as simple as possible by using the $\frac{3}{2}$ power of the head, instead of causing the head to vary according to some other fractional or decimal power, which would require the use of logarithms, logarithmic paper, or a logarithmic slide-rule for practical computations.

Judgment based on experience is one of the best checks on computations, and is often a valuable criterion as to the reliability of experimental results, and, for this reason, the writer prefers to compute experimental results in the first instance in some manner that will make such results commensurate with some established scale or "yard-stick", as the Francis formula for weirs, or the Chezy-Kutter formula for channels.

If the results appear reasonable, they may afterward be applied to any formula desired. Francis applied an exponential formula to the Merrimac River Dam.

The writer early recognized the value of the exponential formula for certain forms of weirs, and applied it where it seemed to fit. Later, Mr. Philip Parker, using the writer's computation of the Bazin, U. S. Deep Waterways, and U. S. Geological Survey experiments, deduced exponential formulas for a large number of weirs.†

The writer is not in sympathy with the practice adopted by some, of working out a table of multipliers to be applied to the calculated discharge over a thin-edged weir to obtain the discharge over an irregular crest-section. First, the discharge over the weir in question might as well be calculated in the first instance, thus saving a deal of computation, with chances for error; second, for a given head and cross-section, the velocity of approach correction varies with the discharge coefficient, so that for two weirs, one thin-edged and the other of irregular section, with the same measured head, but with different sections of approach, the use of multipliers becomes confusing and may be a source of error.

The use of either the Francis base formula or the exponential formula, both with varying coefficients where necessary, seems to the

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* The presence of *points d'arret*, cusps, etc., in weir coefficient curves is shown very clearly in some of Bazin's experiments. See the writer's discussion of the paper by the late George W. Rafter, M. Am. Soc. C. E., entitled "On the Flow of Water Over Dams". *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 345.

† A few of Mr. Parker's formulas appear in his recent work "The Control of Water."

writer better practice, for cases where there is no abrupt change in form of nappe, as when the water breaks free from a flat crest and assumes the thin-edge weir form, the exponential form of expression probably gives in many cases the simplest and most complete history of the discharge. Mr. Horton.

In conclusion the writer is prompted to remark, regretfully, as regards hydraulic science, "Of the making of many formulas there is no end."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.

Discussion.*

By N. T. Blackburn, Assoc. M. Am. Soc. C. E.

N. T. Blackburn, Assoc. M. Am. Soc. C. E. (by letter).—The writer was introduced to the practical side of the stadia method of surveying immediately after graduation from college, and has had a good deal of experience in its use along the coast of Texas, under the U. S. Engineer Office. Its adaptability and flexibility are so great that he has used it almost entirely, except for hydrographic surveys which take in large areas some distance from shore. Where two transits or sextants are not available for locating lines of soundings by intersection, and the lines do not run too far out from shore, a very good survey can be made by locating points on the lines of soundings with a stadia rod held in the sounding boat. Mr. Blackburn.

The writer agrees with the author as to the use of fixed stadia wires and the disregard of the $f + c$ factor. Where carefully measured base lines are run by this method, taking short distances, when the atmospheric conditions are favorable, and with great care in reading the rod, its use may be advisable. For ordinary work the $f + c$ factor is certainly a useless refinement, particularly on side shots. The writer has never attempted to read the rod closer than the nearest 5 ft. on side shots of any length, and he has never been able to guarantee that his side shots in open country, in the middle of the day, for distances of more than 1 500 ft., were within 15 ft. of the correct distance, on account of the trembling of the air. With ideal

* Continued from December, 1913, *Proceedings*.

Mr. Blackburn. weather conditions—a cool, cloudy day with no wind, or very light wind—and short shots, it may be possible to read the rod close enough to allow a correction for $f + c$; but the writer's experience has been that it is generally impossible even to graduate a rod to fit the instrument exactly the same at two different times in the day.

On his first work the writer used the ordinary diamond diagram for the stadia rods, Fig. 2. This marking is plain and easily read except that in the hot sun in open country and at distances of 1 000 ft. or more, the red triangles tend to blur, and the dividing line is hard to distinguish. Moreover, any rod which has the same diagram for each 1-ft. interval is confusing to read. The red figures, however, are an advantage in woods or brush, as they show up in contrast to the foliage and undergrowth. A better form of marking is shown by Fig. 3. This was given to the writer by a member of this Society who was connected with the U. S. Reclamation Service at one time, and is understood to have been used by that organization. It will be noted that only every fourth foot is graduated to tenths. One stadia wire is set on a dividing line between foot marks so that the other wire falls within the divided block. Distances can be thus read more rapidly, particularly for long shots, as the length subtended between stadia wires can be read in 400-ft. blocks.

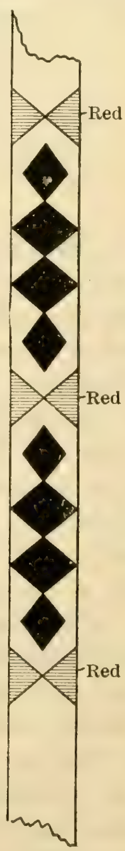

FIG. 2.

FIG. 3.

A rib on the back of a rod is useful to prevent warping and to furnish a handle by which to hold it, but it adds to its weight and complicates its use as a line rod when taking azimuth. It has been the writer's experience that stadia rods warp badly unless carefully watched.

The author's tables showing errors of closure for stadia traverses are interesting and instructive. On a survey of Buffalo Bayou, Texas, a base line was run along each bank by transit and stadia, and the two lines were tied across at intervals of from 1 to 3 miles, thus forming a series of closed traverses from which the errors of closure could be computed. About six of these closures were thus made, and it was found that the error varied from 1 in 155 to 1 in 2 200. The distances on the 1 in 155 traverse were afterward chained and the error reduced to 1 in 1 930. The stadia work was done very carefully, and all distances were read forward and back, but no $f + c$ correction was made.

Mr. Blackburn.

In keeping notes the writer has always numbered consecutively his stadia hubs, and also all side shots from each hub. Then any side shot is located by a double number, first, that of the stadia hub from which it was taken, and then that of the side shot, thus: 15-1, 15-2, etc. A page of notes would appear as shown by Table 24. True south is taken as zero and true north would then be 180 degrees.

On the right page is made a rough sketch of the area surveyed, and the side shots are spotted thereon with their number placed alongside. A note is also made, on the same line as the notes, as to the object on which the side shot was taken. This method has always been clear to the draftsman. As a usual thing, it is better for each man to plot his own work, but this is not always possible, particularly if it is to be plotted as fast as taken. Where the rodman remains always some distance from the instrumentman, and is able to sketch fairly well, it is usually a good practice to furnish him with a book in which to sketch the topography and locate the points on which he has held the rod.

TABLE 24.

At.	On.	Distance, in feet.	Azimuth.	Magnetic bearing.	Vertical angle.	Elevation.
15	14	563	12° 20′	S. 4° 30′ W.	....	....
(H I. = 15.4)	16	750	220° 49′	N. 32° 50′ E.	....	....
....	1	530	270° 50′	....	5° 10′	....
....	2	650	155° 13′	....	....	....
16	15	750	40° 49′	S. 32° 50′ W.	....	....
(H I. = 17.2)	17	920	187° 15′	N. 0° 45′ W.	....	....
....	1	220	192° 00′	....	....	....

The writer has always used with success the method of observation on Polaris for azimuth given in Johnson's "Surveying."* However, it is plausible that the heat of the light used to read the verniers might affect the instrument, and he is glad to know of this.

During the winter of 1911-12 the writer organized and supervised a topographical survey of the Guadalupe River, Texas, on which methods similar to those described by the author were used, although not so accurately in some particulars. This survey extended over about 38 miles of the river, from a point about 14 miles above the mouth to the Town of Victoria (Mile 52), and covered not only the bed and

* Page 558 *et seq.*, 15th Edition.

Mr. Blackburn. banks of the river, but an area about $\frac{1}{2}$ mile back on each side. All distances were measured by stadia. A base line was run along the bank of the river, crossing from one side to the other wherever necessary to get the most open country. On this base a double-rodded line of levels was run very carefully. This base line then formed the horizontal and vertical control for the entire survey. The meanders of the river and all topography between the water's edge and the top of the bank were taken directly from this line. The topography back from the river bank was obtained by running random lines from points along the base. These were similar to the banco traverses described by the author. Most of the elevations for topography were obtained by level, running alongside the transit. The transitman located the controlling points, and the levelman obtained the elevation, which was immediately given to the transitman and entered in his notes. This method requires more instrumentmen, but it is very rapid and simple, and saves much time in the reduction of stadia notes. Where it would have required several "set ups" of the level to obtain the elevation, resort was occasionally had to vertical angles. The true meridian was obtained from a magnetic base established by the U. S. Coast and Geodetic Survey at Victoria, and was checked at a place about 14 miles below the starting point by direct observation on Polaris by the method previously referred to. The field work of this survey cost about $125 per sq. mile. This high cost was due to the bad weather encountered and to the thoroughness with which the work along the river was done. Much of that work has not been described herein. The cost includes also the purchase of considerable equipment, fitting up a houseboat, etc. The survey was made for the purpose of preparing an estimate and report on the improvement of the river by locks and dams. For the area back of the river bank the cost per square mile was probably less than one-half the figure given.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

By Messrs. Hiram F. Mills and T. U. Taylor.

Hiram F. Mills, Hon. M. Am. Soc. C. E. (by letter).—The writer is unable at present to make a critical study of this subject, but would like to ask Mr. Hazen if he has sufficiently provided for the condition —observed many times in New England rivers—that a very small run-off is often followed by a second year of small rainfall, which gives more trouble because following after ponds and ground-water were lowered? Mr. Mills.

Exceptionally, we have the third very dry year, as in 1911, when storage reservoirs were generally empty, and very great anxiety was relieved by a moderate rain in October and more in November. This third year, of course, he would regard as the exceptional year, when emergency supplies or borrowing may have to be resorted to; but the writer is not quite sure that the second very dry year—which often occurs—is provided for.

T. U. Taylor, M. Am. Soc. C. E.—Among the elements or factors entering into the problem of the storage of water for municipal water supply, the author mentions the following: Mr. Taylor.

1.—The size of catchment area or water-shed;
2.—The mean annual run-off per square mile;
3.—The portion of water area and the loss by evaporation from it;
4.—The natural storage in lakes, or deposits in sands, gravels, or pervious materials;
5.—The regularity of flow.

* This discussion (of the paper by Allen Hazen, M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, and presented at the meeting of December 17th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Taylor.

Having spent more than half a century in the West, and having been more than half that time a close student and observer of water conditions in the great southwest section of the United States, the speaker welcomes any discussion of storage problems, even though the data for the most part refer to sections of the country with conditions unlike those of his own. Such discussions are valuable, although they may serve the sole purpose of convincing engineers that the data for the East cannot be applied to the West, and that the man who does this brings disaster on the enterprise and should bring it on himself. However, such discussions are valuable, whether for municipalities, irrigation, or power. In the Southwest, especially in Texas, all these problems have to be considered, and it sometimes happens that two, or perhaps all three, storage problems may occur on the same river.

Two of the elements entering into the problem, as mentioned by Mr. Hazen, can be omitted in Western or Southwestern problems, namely, the water area in the water-shed and the natural storage in lakes. The three fundamental factors in Texas are:

1.—Area of water-shed;
2.—Regularity (or irregularity) of flow;
3.—Run-off.

The area of the water-shed can be ascertained with a reasonable degree of accuracy, and the regularity of flow can, in general terms, be ascertained by local testimony—and it is about the only factor in the problem where the testimony of the oldest inhabitant is worthy of serious consideration. For accurate calculations, however, it would be necessary to have hydrographs constructed from daily records of flow, and these can only be obtained by gauging stations established and maintained with unbroken records of gauge readings and of annual rating curves. The third factor, the run-off, is the one which contains possibilities of the gravest error. Any attempt to apply run-off data obtained in one section of the country to a locality or stream in another section is likely to result in great damage, unless Providence is kind. It is full of the utmost danger, as many wrecks in the West will bear silent and bitter testimony. The regularity of flow, the quantity of daily flow, the mean monthly flow, the mean annual flow, and the run-off are data that can be obtained only by careful observations and measurements for the locality, station, and stream concerned. The run-off for the same stream, at different stations along its course will be found to vary greatly. The speaker calls to mind a case where the run-off on a stream at one point is 0.1 sec-ft. per sq. mile of water-shed, and at a point 1 mile above it is only 0.01 sec-ft. A stream may course its way through a coastal plain of a reasonably flat country, then enter a canyon section where

floods occur with torrential violence, and then again, in its upper reaches, it may enter into a flat upland section. It will be found, and, in fact, has been found, that the run-off and flow in second-feet per square mile of water-shed differ greatly. The problems in the West are largely concerned with the rainfall, run-off, mean flow, and the necessary storage capacity to equalize the flow or to equalize it sufficiently for the municipality, irrigation project, or power problem. Mr. Taylor.

Careful daily records and observations on the Colorado River of Texas, at Austin, Tex., for the last 14 years, have shown that the average or mean flow for this period has been 1 820 sec-ft. The water-shed of 37 000 sq. miles would give a mean flow of $\frac{1}{20}$ sec-ft. per sq. mile. The lowest mean annual flow for this period was $\frac{1}{74}$ sec-ft. per sq. mile, and the largest flow on record, occurring at the time the Austin Dam failed, April 7th, 1900, amounted to less than 4 sec-ft. per sq. mile. The lowest flow on record occurred from August 15th to 20th, 1910, when the run-off was about $\frac{1}{1800}$ sec-ft. per sq. mile.

The mean annual rainfall on the water-shed of the Colorado above Austin for the period was practically 24 in., but the mean flow for the period was only 1 820 sec-ft., which is equivalent to a run-off of 2.88%, or 0.7 in.

With these staggering facts available, a $4 000 000 enterprise was established in a neighboring territory, when, after unusual rains, the engineers awoke to the fact that, in using an assumed run-off derived for other regions, they had omitted the most vital factor, and had, according to their own admissions, over-estimated the possibilities of the scheme by a large percentage. The money had been spent, the water-shed was there, and the rains came, but the run-off refused to run with the vigor that it was assumed to display on such occasions. The only plea of defense is that the engineers were misled, as to the run-off, by a book on irrigation, when the book made no statement as to the run-off in the territory mentioned.

A large municipality authorized the construction of an impounding reservoir and its appurtenances at a cost of $1 000 000, on an assumption of a run-off of 19%, and again there was a failure to come up to expectations.

The run-off is a complex factor, depending on topography, vegetation, kinds of soil (whether cultivated or uncultivated), rainfall, distribution of rainfall as to time of year and as to growth of vegetation, and, what is still more vital, the condition of the soil at the time of the rains. Personal observation has convinced the speaker that a 2-in. rainfall in 24 hours may at one time give a run-off of 25% and at another time no run-off whatever. In 1910 there was the lowest flow of the Colorado River at Austin, but it was not the year of least rainfall, nor the year of lowest mean annual flow. Each stream is a

Mr. Taylor. problem unto itself. An experienced engineer will go very slowly in such matters. Like Davy Crockett, he will be sure he is right, and then go ahead, and not, like many, "rush in where angels fear to tread."

The Colorado of Texas has a canyon section or power section, a municipal section, and an irrigation section. At present, due to a lack of storage, neither the power section nor the irrigation section is reliable as a revenue producer, as the flow is spasmodic and irregular. The construction of storage reservoirs on this river is a simple and entirely feasible problem, and it would be an easy matter to store about 20 000 000 000 cu. ft.; and this, in the driest years, would give a minimum flow of 800 sec-ft., which, with the total head of 240 ft. that could be obtained by the various dams, would give about 20 000 h.p. Then, in addition, this reliable and equalized flow would insure the irrigated rice crop 250 miles below, near the coast. Altogether, the Colorado offers one of the best problems for storage known to the speaker. Its records as to flow, area of water-shed, run-off, dam sites, and the demand for power and for reliable flow for irrigation, have all been worked out, and there need be no assumptions as to any factor.

The method suggested by Mr. Hazen for rating the dry years is safe, and would fit any section of the country, because each river would be writing its own biography. It might sound a little more comforting to the people of the West if the percentages were given in "wetness" instead of "dryness". The dryness feature is obtruded on them with sufficient emphasis already, because it is a land

"Where the prairie dog kneels,
On the back of his heels,
And silently prays for rain."

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.

Discussion.*

By Messrs. W. J. Wilgus, J. E. Willoughby, Frank C. Boes, Allen Hazen, H. C. Vensano, Leonard Metcalf, William B. Jackson, F. Lavis, Alexander C. Humphreys, Henry Floy, Clinton S. Burns, Stuart K. Knox, and J. H. Gandolfo.

W. J. Wilgus, M. Am. Soc. C. E. (by letter).—The distinction made by Mr. Alvord, that the formal setting aside and labeling of depreciation reserves should entitle the owner of a property to a return on the full cost of its reproduction new, and that the non-observance of this technicality should deprive him of that right, does not appeal to the writer as being just or logical. Surely, if the general balance sheet of a corporation shows an excess of assets over liabilities sufficient to offset the accrued depreciation of the property, there would seem to be no more reason why the return should be based on an impaired investment than that the rent paid by a tenant should be lessened if the owner, who has agreed to keep the premises in repair, elects not to maintain a special fund therefor in the bank. Mr. Wilgus.

The majority of railroads have ample profit and loss surplus accounts invested in outside securities or in the property in excess of the amount on which a fair return is earned; and this surplus, for all practical purposes, is a depreciation reserve which insures the ability of the corporation to meet its liability of paying for maintenance out of income. A failure by a corporation properly to meet its liability

*This discussion of the paper by John W. Alvord, M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, and presented at the meeting of December 17th, 1913, is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Wilgus. of maintaining the property out of the rate, is usually the result of over-payment of dividends, and eventually brings its own punishment through the necessity of a reduction or suspension of dividends during the period of rehabilitation. This course is obligatory, as charges for rehabilitation legally can only be made through income.

Whether or not a depreciation fund is actually set aside would seem to be immaterial. What the public is interested in is the proper upkeep of the property without an increase, for that purpose, of the capital burden which it is expected to bear through the rate.

As Mr. Alvord states, "* * * no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand"; but to determine whether or not it is in hand may be done just as effectively through an examination of the balance sheet in conjunction with a valuation of the assets, as in looking for a labeled fund in the bank. To "fine" the owner who elects to protect his liability in some other form than by means of a special fund, seems to be rather a drastic punishment for his selection of one of two alternative treatments of an accounting detail.

Mr. Willoughby. J. E. Willoughby, M. Am. Soc. C. E. (by letter).—Few engineers will disagree with Mr. Alvord's suggestion,

"That if a sinking fund * * * for depreciation is actually kept * * * in trust as part of the property, it should * * * receive the same rate of fair return as the remainder of the property 'used and useful for the public.'"

The writer, however, believes that this suggested addition to the capital account will find little favor. Depreciation is an operating expense when applied to the accounting of a going public utility, and there is no more reason for capitalizing the depreciation item of operating expenses than for capitalizing any other single item, for instance, the wages paid to enginemen in charge of railway trains.

Furthermore, such funds have not been generally provided as a part of the capital account of our public utilities, and we gain directness by discussing the actual conditions, and the need of providing for depreciation as an operating expense, so that the accounting will always show the value of the public utility for any purpose, rate-making or otherwise.

The accounting of a public utility covers other things than the cost of the plant, and it is the ultimate finding of the accounting (exclusive of the speculative and intangible features) that fixes the value of the utility. The author, however, limits his discussion to that "line of evidence as to value known as the reproduction method, or cost new less depreciation." By thus disregarding all other elements of value that enter into a public utility property, the author has no credit account to offset depreciation, except the cost of reproduction

Mr. Willoughby.

new of the physical property of the utility, and he would limit the rates to be charged by the utility corporation to a reasonable return on the capital necessarily expended for a new plant less the accrued depreciation. This means that the utility corporation must, immediately after reproducing its property new, write off as loss the amount of the accrued depreciation, although that accrued depreciation has actually resulted from service rendered to the public, and, although the only practical way of reproducing the physical property will be to invest first capital to the extent necessary to reproduce the property new.

It is generally accepted that the reproduction of the physical property of a public utility which has been in operation for some years so as to show its exact stages of depreciation would be at a greater cost than to reproduce it with all new materials. Consider the difficulty and expense of providing and assembling into a completed structure the curve-worn rails, the partly decayed ties, the partly fouled ballast, the partly failed bridges, trestles, and culverts, and the argument is completed. Consider then the injustice of the suggestion of the author that the property owner should be penalized because of the impossibility of assembling his plant, except as a structure composed of new elements. What the public pays for is service. If the service is satisfactorily performed, as in the railway after some years of operation, over rails, cross-ties, and ballast, each exhibiting but 55% of their cost new, less salvage, and if to perform that service it has been necessary for the owner to expend in the past 100% for those items, the owner should receive a reasonable return on that 100 per cent. The public's protection lies in scanning the operating expenses to see if the charge for maintaining the property is unduly high, as a result of the failure of the railway company to make provision in the past for the depreciation at the time the depreciation actually occurred, and not to deduct from the capital originally expended the amount of the accrued depreciation. It is only after the operating expenses are deducted that any return can be made to the owners of the stocks and bonds. If the operating expenses are unduly high, on account of failure to make provision for the depreciation as it had accrued in the past, the penalizing of the owner occurs in the denial of deduction from the gross earnings of so much of the alleged operating expenses as is due to the failure of the owner to provide for the depreciation at the time of its accruing. The method of penalizing the owner, as suggested by the author—the deduction of the accrued depreciation from the reproduction cost new—is a confiscation of property to the amount of the accrued depreciation, because the amount of the accrued depreciation was originally and necessarily put into the property before any depreciation could take place, and whatever has taken place

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is properly to be regarded as an expense of operation. No engineer would deduct from the physical elements of a property the operating losses due to bad financial arrangements, extravagance, or incompetency.

To illustrate further: take a cross-tie costing 80 cents in place, and lasting 8 years. For each year following the placing of the tie, there should be charged out as operating expense the sum of 10 cents (less by such interest as it may earn), as the proportion of the cross-tie consumed in that year's operation, so that when the cross-tie is replaced in the eighth year only 10 cents of its replacement cost would be charged into operating expenses, the other 70 cents being taken from the depreciation fund. The purpose of the depreciation fund is to distribute the operating cost of renewal over the number of years that has been taken to consume the tie. Where a depreciation fund has not been accumulated to offset the actual depreciation, the monetary value of such depreciation will go into the profit and loss account as a debit, but there may be such values in that account—like accumulated reserves—which will entirely absorb the debit. Depreciation, when regarded as an operating expense, can never be used to lessen the reproduction cost of a public utility. It will always affect the value of the utility as a going concern.

Every public utility should make provision in its accounting for depreciation, because such an account is necessary to show the annual operating cost of any plant, and to reflect in the accounting the financial and physical condition and value of the plant. It should cover "all the losses of value that occur in the property, plants, or parts thereof, from wear and tear, obsolescence, or inadequacy."

For a public utility that has permitted the depreciation to accumulate without providing for an actual fund to offset the same, then the amount of such accrued depreciation (in such event a debit) will go into the profit and loss account, there to be offset by the credit items of other accounts. Correct accounting does not require the actual presence of cash, except as to the amount of the item of cash on hand.

As to what rates are necessary for a fair return for any public utility, if confiscation is not to result to the property in the legal sense: the suggestion is made that the rates should be sufficient to provide for:

1st.—The payment of taxes and other public dues;

2d.—The payment of all operating expenses, including as a part thereof a charge sufficient to care for the annual depreciation that takes place;

3d.—The payment of a reasonable commercial interest (commercial interest being one that considers and includes the element of risk involved) on the reproduction cost new of the property, including the necessary working capital;

4th.—The payment of a like interest on the intangible values of the property and the unearned increment; Mr. Willoughby.

5th.—And, finally, the accumulation of funds sufficient to provide for the non-earning additions demanded by the public. The commercial interest earning additions and betterments can be capitalized and becomes a self-supporting part of the cost of the plant.

FRANK C. BOES, ASSOC. M. AM. SOC. C. E. (by letter).—After reading Mr. Alvord's paper, the writer tried to picture to himself some of the possible results of putting such a system of rate-fixing into general operation. Mr. Boes.

Consider two adjoining and otherwise similar communities served by two power companies, the one having an old and decrepit plant, but still capable of rendering efficient service, and the other having a new and up-to-date plant. The large users of service, of course, would gravitate toward the community served by the depreciated plant, thereby decreasing property values and rents in the community which they left, and making it impossible for the new plant to compete with the old one. The continual see-sawing of rates, due to periodic repairs and renewals, would make it impossible for service rates to reach a standard level, thus making it impossible for a consumer to foretell what his next year's bill for service would amount to, and would also cause much litigation and confusion.

To fix rates purely on a basis of physical value, would remove incentives for good management and economy, and put a premium on wastefulness, expensive plant, and unnecessary repairs and renewals. Over-expenditure on plant, for the sake of keeping the rate up, is more to be feared than a reasonable amount of decrepitude, for, after all, the service rendered is what is paid for.

If such a system were in vogue now, those who live on Staten Island would be paid a substantial sum for riding on trolley cars which have long outlived their years of usefulness and are run by a plant which is in the condition of the "one-hoss-chaise." They are consoling themselves by the thought that they pay, not toward earnings on a dilapidated outfit, but for the service rendered—a 5-mile ride. A company owning a modern plant might give worse service.

Mr. Alvord states that the average life of the component parts should give the composite life of the plant. This must be an oversight on his part, for, assume a plant to consist of two parts, A, costing \$10 000 and having a life of 5 years, and B, costing \$5 000 and having a life of 20 years. Surely the composite life of the plant is less than $12\frac{1}{2}$ years, for its major part is renewed twice in that time.

In fixing rates by such a method, one must also consider that the depreciation curve is not a straight line. It is permissible to establish

Mr. Boes. a depreciation fund by the straight-line formula, for that is merely a convenient method by which the owner may distribute his renewal costs evenly. The amount of the depreciation fund by no means represents the actual physical depreciation, and to base the deduction of rates on an amount which is taken arbitrarily, and is greater than the actual depreciation, would be unjust, particularly if the service were still unimpaired. When a laborer reaches old age, his earning power is greatly reduced, but is he worth less at thirty-five than at twenty? One cannot expect to deduct $3 000 from a $10 000 plant, in fixing rates, and still have a $10 000 service maintained. If the returns are not based on the entire investment continuously, the investment will not have renewed itself at the end of the plant's term of life, and the owner will not have the plant that he had to begin with. Depreciation is merely another form of capital, and must be kept continuously earning returns on its entire amount.

It seems unjust to expect to pay less for service in the fifth year, the service being exactly the same, merely because some parts of the plant will require replacement at a date then nearer than in the first year. Does one pay less for milk given by old cows or for apples grown on old trees?

Mr. Alvord intimates that the reduction in rates should be considered in the nature of a fine, but surely it would be unjust to have a fine imposed, if, when the time for renewals actually arrived, the owner produced the funds and put the plant in shape. What then? Would the amount of the fine which he had paid be returned to him? A man cannot be fined because it is thought that he is going to commit a crime.

An argument sometimes advanced, and by the Courts, is that one should not make one valuation for rate-making purposes, and another valuation for the purpose of sale or purchase; that is, if valuation is deducted in making a sale of a plant, it should be deducted in making the rates of service. Consider some private business: If an owner wishes to sell his factory, he must certainly make a deduction for depreciation, but the price of his merchandise is not affected in any way by the age of his factory or by the condition of his plant. Neither do his customers inquire as to whether or not he has an adequate renewal fund actually in hand. That responsibility rests with himself alone.

Though there is a difference between fixing the prices in a private business, and fixing the rates of a public utility which enjoys monopolistic privileges granted by the public, the fundamental principles remain the same in each case.

The approaching repairs and renewals are an obligation and a cause of worry to the owner, and to him alone. He has to pay for them, and

the consumer is in no way concerned with it, as long as the service remains the same. The consumer is not concerned with how near the day of collapse the plant may be, as long as the collapse never occurs. The consumer is concerned only with impairment of service. When that occurs, then a fine may reasonably be imposed. Mr. Boes.

In fixing rates, one need only be sure that the property is in satisfactory working order and that proper service is being given. The owners can then be left to worry about the depreciation for themselves.

Allen Hazen, M. Am. Soc. C. E. (by letter).—Depreciation has to be taken into account in both the operation and valuation of public service properties. It has to be taken into account because it is one of the fundamental facts that cannot be ignored. Mr. Hazen.

The writer is familiar with depreciation, especially in water-works properties, and what he has to say relates primarily to them.

Allowances for depreciation have actually been made in nearly all properties of this kind. This is not the less true because the amount allowed has not been determined in a rational manner, and frequently has not even been called depreciation. In most public works the first cost has been paid by money raised by the sale of bonds, and a sinking fund has been established, and annual contributions made to it, intended to be sufficient to provide for the payment of the bonds when they became due. This, in reality, providing for depreciation, even though the word "depreciation" is not used.

Another way of providing for depreciation which has been common, has been to pay for new construction out of earnings. This also is an efficient means of allowing for depreciation, although the methods and amounts to make it efficient have seldom been carefully thought out. In general, with publicly owned properties, the allowances that have actually been made for depreciation in these ways have been above the truth. This has been fortunate, as the excess has represented only a slight addition to rates which have not been burdensome, and the resulting strong financial condition of the works has tended to good service, worth more than its cost to the takers.

In the relatively small number of cases where no allowance, or an inadequate allowance, for depreciation has been made, there results sooner or later a cramped financial condition tending to bad service, and equally unfortunate for the works and for the takers.

The measure of depreciation for any period is the decrease in the fair value of the property under discussion, or any part thereof. In valuation proceedings, where the cost of reproduction is being considered, the decrease in the value of the property, as compared with the value of corresponding new property, is the measure of depreciation. The fair value of the property is to be considered always as the fair value between a willing seller and a willing buyer. To take

Mr. Hazen. a concrete case, it may be considered how much less a willing buyer would probably pay and a willing seller take because the pipes in the streets had been in use for a time, and had suffered the depreciation naturally incident to use. Obviously, some deduction would be made. The carrying capacity of the pipes is less than that of new pipes, owing to rusting and tuberculation; there is more leakage from the joints in old pipes than in new ones, representing loss of water and profits. It may be presumed that the pipes in an existing system laid from time to time are perhaps less efficiently arranged as to sizes and locations than would be the case in such a pipe system as the buyer would lay for himself. These would probably be the most important points to be considered as between the buyer and the seller in deciding what deduction was to be made. In addition, any peculiar conditions of any kind affecting the value of the property would be duly considered.

The average age of pipes in any system in a growing city will ordinarily be from one-third to one-half of the age of the system, and seldom more than 15 or 20 years. The bare fact that an average of 15 years has elapsed in a possible ultimate life of from 75 to 150 years, which is assumed by the author, would have but little influence between a willing buyer and a willing seller.

The case is somewhat different with boilers, which may be assumed to have a life of 20 years, or some other definite period, and will then have to be replaced. The boiler represents the exceptional case in water-works affairs, and not the ordinary one.

The writer believes that there is no such thing as a period that can be called the life of cast-iron pipe. Different pieces of pipe may have useful lives ranging from a few hours to periods so long that they extend beyond our experience. Any statistics that could be collected would contain records of pipes that had been discarded for one reason or another, and would contain, at best, only estimates of the remaining life of the very important part of all the pipe laid that is still in service. The estimates of the remaining life of this pipe cannot be considered to have much value, and averages based on them are certainly no better. Even if the average life of pipe could be ascertained in some statistical way, it would be of little service. What is really wanted is the average percentage payment on the value of all the pipe to be made each year to make good the depreciation lost in it, and this percentage is undoubtedly quite different from the depreciation computed by the sinking-fund method from the average life of pipe, if it could be ascertained.

The writer believes that in the case of cast-iron pipe the gradual reduction in carrying capacity, the increase in leakage from the joints, and the fact that a system now designed might better serve its pur-

pose than an old system, are the dominating factors, and that the elapsed period of life is of secondary importance. Mr. Hazen.

It may be suggested that in using the sinking-fund method it is possible to assume a life for pipe to give any rate of depreciation that is desired, and as the assumed life is pretty sure to be beyond the experience of the ordinary water-works system, it cannot easily be shown to be fallacious. Thus, with a 4% sinking fund, the depreciations corresponding to several lines are as follows:

30-year life,	1.78%	annual depreciation
40- " "	1.05%	" "
60- " "	0.42%	" "
80- " "	0.18%	" "
100- " "	0.08%	" "

Of course, the percentages given refer to the structures only when new. By the sinking-fund rule they would steadily increase with increasing age.

The ease of selecting a life to give any desired rate of depreciation is obvious. Considering the great difficulty, or perhaps rather the impossibility of securing statistics to show the true average life of cast-iron pipe, it may be questioned whether, as a matter of fact, the process has not been reversed, perhaps unconsciously, by estimators, and a life selected which would give a rate of depreciation which has seemed to them reasonable, as judged by other standards. If this is really the case, then the use of the sinking-fund method for computing depreciation may be regarded as a kind of mental exercise tending to familiarize the estimator with the workings of compound interest during long periods, and thereby to aid in keeping him from reaching unreasonable conclusions.

The sinking fund has this to recommend it: that it corresponds with practical experience to the extent that depreciation in the early life of a structure goes on slowly, and that as the years go on the rate is increased. There is a tendency to substitute a rule of calculation for judgment, and, as far as the basis of the rule can be well established, this is desirable; and, if a fixed rule is to be used, probably the sinking-fund method of computation is the safest that has been thus far proposed, but it must not be supposed that the real basis of the sinking-fund method is the actual determined life of the structures. It is, instead, the life which practical appraisers find will give rates of depreciation that are in accordance with their experience.

The writer believes that the most logical way to estimate depreciation in the appraisal of a plant is to consider each item on its merits, to determine and take into account its physical condition, its usefulness in the service and the state of the art, and the possibility of substituting other and more efficient structures to perform the ser-

Mr. Hazen. vice rendered by it. The excellence of the design must be especially considered, and depreciation may be several times greater on a clumsily designed structure than on a well-designed one, although the durabilities and lives may be the same. In a similar manner, a well-built structure suffers less depreciation than one not so well built.

Improvement in methods of pumping, filtering, laying pipes, or performing any part of the service, all have their bearing on the depreciation of similar parts of an old plant. The rule must always be to find what deduction would fairly be made because of all the conditions that exist between a willing seller and a willing buyer fully informed as to those conditions.

In estimating the amount of depreciation to be written off each year, the writer believes that one of the best kinds of information is to be obtained from the financial histories of old plants in connection with appraisals. The question may then be taken up as to whether an annual depreciation of 1% on the cost of reproduction of the property from year to year, made during the whole life of the plant, would have served to have brought the book value to an amount reasonably near the estimated cost of reproduction at the time of the appraisal, and if not, to find as nearly as may be what percentage thus allowed annually would have sufficed to do this. Data of this kind are hard to get, but the engineer having to do with appraisals occasionally has opportunities to secure them. Valuable as these data are, they must be used with caution, as the depreciation of the future is not necessarily measured by that of the past.

The method of allowing for depreciation in the annual operating account is an important matter. The writer recalls the case of trustees who had undertaken the management of an important water-supply system, and who were confronted on the one hand with the difficulty of raising money for urgently needed extensions, and on the other of being required by law to take up a business entirely foreign to that for which they were appointed, namely, that of setting aside a certain proportion of the income from the operation of the property and investing it in securities to be held against the redemption of bonds when they became due. The situation was a trying one, and one that ought not to have existed.

The sinking fund was invented to protect the bondholders who might think that otherwise the city would not be able to raise so large a sum of money on the maturity of the bonds. That is the only possible justification for its existence. If we grant the solvency of the city or company, and assume that they will be able to refund the bonds when they become due, it is better in every way that the allowance for depreciation, or the sum that would otherwise be put into a sinking fund, should be invested in the plant as far as there is need of it.

The author does not know of any sinking fund that has been established for the especial purpose of meeting depreciation. Such a fund as he suggests would be one to which annual depreciation allowances would be paid, and from which money would be taken to replace worn out or discarded parts, and the amount of such a fund, he thinks, would not often exceed 10 or 15% of the value of the plant. The writer feels that the question of whether such a fund is necessary or desirable must be left to those who operate the plant. If it is desirable, he suggests that it might be more appropriately called "working capital", as its function would be quite different from that ordinarily covered by the term, "sinking fund". Mr. Hazen.

It is the writer's feeling that, so far as such a fund of working capital is reasonably necessary and desirable for the proper and economical administration of the plant, the amount of such fund might properly be taken into account in fixing the rates that might be charged without exceeding those that would yield a fair return on the property; but he sees no reason for taking into account otherwise any part of the value of the original plant that may have been lost by depreciation in making such rates. It is clearly the privilege and duty of public service corporations to charge rates that will permit marking off a proper allowance for depreciation. When this allowance has been once determined and marked off, there would seem to be no equitable reason why the capital thus marked off should be further considered in connection with the question of fair rates.

H. C. Vensano, Assoc. M. Am. Soc. C. E. (by letter.)—in looking over this interesting paper, the writer was surprised to find the argument that, unless a sinking fund for depreciation is actually in bank or in trust, it should not be considered a reason for not deducting depreciation in valuations for rate-making purposes. Mr. Vensano.

In his summary, the first and second arguments indicate that Mr. Alvord feels that even if a sinking fund exists, reinvested in the plant itself or elsewhere, it should not be considered as such.

To the writer it seems axiomatic that, if a sinking fund, properly handled and accounted for, exists anywhere at all, its existence is a good reason for the non-deduction of depreciation in valuations for rate-making purposes.

If a sinking fund to cover depreciation has been provided, and if it has been properly handled, and if, as a matter of course, the returns from such fund, in no matter what form they are received, have been turned back into that fund, it would seem clear that it is immaterial whether such fund exists in bank and trust, in the property itself, or elsewhere.

Mr. Alvord seems to feel that, even with proper funds reinvested in the property and accounts properly kept, such accounts are merely

Mr. Vensano. the opinion of the owner as to depreciation which has accrued on his property, and further argues that when a fund is handled in this manner the owner will have a return on it as a sinking fund and "at the same time have a return on it as a betterment to the plant," and that "this would be an obvious injustice to the public."

If the fund is properly kept, the accumulations from such investment, of course, will go into the sinking fund itself, and no portion of it will be returned as dividends to the owner, and in such case no injustice to the public would follow and no duplication of returns on investment would be made by the public to the owner. The whole point hinges on the proper accounting, and with such proper accounting the public's interest will be entirely safeguarded. As a matter of fact, any sinking fund, whether in bank or elsewhere, is now largely an "opinion of the owner" as to the depreciation on his property, or rather on the rate at which it is likely to occur, and will always represent either the owner's opinion or the opinion of some rate-making or controlling body which he has accepted, or has been forced to accept.

Assume a case in point, and see if any excess burden is thrown on the public. Take a large utility of complex properties, which utility may be considered of such a nature that after a number of years the necessary yearly depreciation replacements would arrive at practically a constant amount (assuming no extensions or betterments as occurring). Assume an original investment of $100 in complex properties having an average life (say, 40 years), such that $0.83 set aside annually as a depreciating increment will produce the proper sinking or replacement fund. Now assume that the property has been in existence so long that annual depreciation replacements have become constant (no betterment or additions considered), so that at this time the sinking fund in bank or elsewhere has reached a total of $33.40, all replacement made in the meantime having been paid from it, and that now the balance has been reached such that the annual increment set aside and the accruing interest on the $33.40 will just pay for the annual replacements, that is, the cost of annual replacements has practically become constant.

Also, for convenience, suppose that the interest rate on this sinking fund (assumed to be in the bank) is the same as the dividend rate to the owners of the utility. Now, in this case (Case 1), the fund is actually in bank, rates of dividend and interest are 5%, and rates have been fixed by a rate-making public service body so that they will provide such dividends plus the necessary depreciating increment of $0.83, as previously mentioned. Table 1 then shows the result.

Now (Case 2), it may also be that extensions and betterments have been made so rapidly that the sinking fund, instead of having been banked, may have been reinvested in the property itself; let it be supposed that this has been done. The public, of course, will furnish

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TABLE 1.—CASE 1.

			Paid by Public.		Paid by Bank.		Available for Replacements and Reserve.		Available for Dividends.	
			Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
Original investment	$100.00	Allowed as profits	5%	$5.00						$5.00
		" for depreciation reserve	0.83%	0.83				$0.83		
Sinking fund in bank	33.40	Interest			5%	$1.67		1.67		
Investment in betterments, through later security issues	33.40	Allowed as profits	5%	1.67						1.67
		" for depreciation reserve	0.83%	0.28				0.28		
Total paid by public				$7.78						
" " " bank						$1.67				
		Total for depreciation reserve						$2.78		
Total investment	$133.40	Total available for dividends							5%	$6.67

TABLE 2.—CASE 2.

			Paid by Public.		Paid by Bank.		Available for Replacements and Reserve.		Allowable for Dividends.	
			Rate.	Amt.	Rate.	Amt.	Rate.	Amt.	Rate.	Amt.
Original investment	$100.00	Allowed as profits	5%	$5.00						$5.00
		Allowed as depreciation increment	0.83%	0.83				$0.83		
Depreciation reserve reinvested as betterments	33.40	Allowed as profits on depreciation reserve	5%	1.67				1.67		
		Allowed as depreciation increment	0.83%	0.28				0.28		
Total paid by public				$7.78						
Total investment	$100.00	Available for dividends							5%	$5.00
		Total to depreciation reserve						$2.78		

NOTE.—The total, $2.78, to depreciation reserve. represents $2.50. the constant increment on original investment (after 40 years life) and $0.28, a depreciation increment on depreciation reserve (Case 2), or investment in betterments (Case 1). This is proportional to depreciation increment originally provided on original investment.

Mr. Vensano. the returns through the rates. The results are shown in Table 2. In Table 1, it has also been assumed that similar betterments have been made through money provided by an issue of new securities.

For proper accounting, if, in Case 2, $33.40 reinvested is still maintained in the accounts as a depreciation fund and its increment of earning is returned to such fund (as would be the case were such fund held in bank, as in Case 1), and if the dividends are then paid as before on the actual investment, exclusive of the sinking fund, it will be seen that the public pays and the owners receive the same amounts in each case.

It must be borne in mind that the annual depreciations on additions or betterments have not arrived at a constant stage as yet, and therefore the same proportionate annual expenditure for replacement is not necessary on such part of the property as on the original assumed properties. It is to be noted, however, that there has been provided and set aside a depreciation increment on the amount spent in betterments, which is exactly proportional to that provided on the original investment. Therefore, if the proper increment, or proper average life for the original investment has been assumed (the additions and betterments, of course, being of the same class of properties), the proper sinking-fund increment has therefore also been provided on such additions and betterments.

This would seem to show clearly that, with correct accounting, sinking funds may be very properly reinvested in the property without extra cost to the public. As a matter of fact, as Mr. Alvord pointed out, the returns on a bank sinking fund will be less than the returns from similar funds reinvested in the property itself, and therefore the public would ordinarily actually be benefited by such reinvestment, in that the depreciation increment, which must be included in the rates paid, could be reduced.

This should show clearly that a reinvested sinking fund is certainly not a matter of mere bookkeeping.

It may be argued that the fund is not as safe when thus reinvested, but the assumption must be made that, under public service rate regulation, the rates will be adjusted upward when necessary as well as downward, so that a constant and safe return will be obtained from public utility investments, and, if this is done, the reinvestment of money in the property will be the safest possible method of handling such funds.

If public utility investments are not made safe under Government regulation, Government regulation will certainly fail as such, and either destroy the utilities or necessitate Government ownership; because, with uncertain interest returns, held at a low rate, no capital will be obtainable for investment in such utility.

It may also be argued that, up to the time where dividend returns on the sinking fund have reached the state of balance and become sufficient to cover the annual depreciation replacement, a fund reinvested in the property is not in such shape as to afford ready use as cash. Mr. Vensano.

It is to be noted, however, as Mr. Alvord says, that large replacements can generally be foreseen a number of years in advance, and a new issue of securities can then be provided to cover such replacement; the betterments constructed under the original sinking fund can be then transferred to capital account under the new securities issued, and the depreciation account credited with the replacement expenditures.

A similar reasoning will hold for any outside investment, provided such investment is safe, and provided the returns from such fund are turned back into it and the whole held as a part of the utility, and, as an additional advisable qualification, that such investment be in liquid form. That is, it might be an investment in bonds of other companies which have a ready sale, and may therefore be considered as liquid. Certainly, no one could object to an investment in Government bonds as being any less safe or less real as a sinking fund than such fund held in bank or in trust.

The writer would also like to add that, in addition to believing that any properly kept sinking fund is a good reason for not deducting depreciation in valuations for rate-making purposes, he also believes that, even in some cases where such fund does not exist, it may be proper to omit the deduction of depreciation. Perhaps it would be better to say that the writer believes that depreciation in general should not be deducted, except in cases where it is clearly shown that the property has been mishandled by the payment of excessive dividends, or mismanaged in some other way, whereby the accumulating of a sinking fund has been prevented, or when such fund has been mismanaged.

Take, for instance, the case of a company which has gone into business recently in proportion to the average life of its properties and equipment. Assume that such company has been in business 10 years, with an average property life of 30 years. In many cases the property might not have paid during its early life, and with the rates at present in force has just arrived at the point where it is able to set aside money for a replacement fund and to pay dividends. Assume that the earnings are sufficiently large to supply a depreciation increment which in the remaining 20 years of life will give the total fund required, which fund, theoretically, should have been started 10 years previously with a smaller depreciation increment. Would it be fair, then, to such company, which has received no return on its investment, and has been unable, because earnings were too small, to provide deprecia-

Mr. Vensano. tion increment, to cut its rates now to a basis fixed by a depreciated valuation. Such property without regulation could no doubt be considered a good and well-handled investment, as business investments go, and still would be placed among the class of failures if its rates were calculated on depreciated valuation, as just shown. In any event, if rates in this case are to be based on such depreciated valuation, it would seem only equitable to make them sufficiently high to afford a depreciation increment large enough to return the entire investment in the remaining life of the plant.

It is admitted, of course, that, in using valuations from which depreciation has been deducted, rates can still be fixed logically and equitably by a proper handling of such depreciated valuation and by allowing earning to cover a depreciation increment based on the remainder of the life of the property, as shown by Mr. Grunsky in the paper to which Mr. Alvord has referred.

The whole question as to whether or not depreciation should be deducted from physical valuations for rate-making purposes would seem to the writer largely a matter as to whether or not the public utility had been handled correctly. It is his opinion that Mr. Grunsky's theory is entirely correct, and, as theory, could be applied to any rationally handled property. It would seem that no theory of any kind can be made to apply to a property which has not been handled reasonably and honestly by its management, and at the same time be made to apply equally to a well-managed property.

Where a property has been earning large returns, and has disbursed them as fast as earned, and over and above a reasonable dividend rate; and where, at the same time, no sinking fund has been provided, it is clear that such company has shown itself ignorant of depreciation results as a whole, and therefore must suffer the penalty.

This, however, would not seem to disprove the theory as such, and as argued by Mr. Grunsky, that, in general, depreciation should not be deducted in physical valuations for rate-making purposes.

Mr. Metcalf. LEONARD METCALF, M. AM. SOC. C. E. (by letter).—Mr. Alvord's interesting paper illustrates anew by what different mental paths different men have reached similar conclusions.

As the writer's views on depreciation are consonant with those set forth by the Special Committee of this Society on the Valuation of Public Utilities in its report which will be in print before this discussion, the writer does not purpose to do more than call attention to certain points in the application of the equal-annual-payment method of figuring depreciation, as compared with the ordinary sinking-fund method referred to by Mr. Alvord. The fundamental difference in the two methods consists in the fact that under the equal-annual-payment method, depreciation is treated as an annual amortization or repayment of capital (renewals *per contra* being treated as new con-

struction), and the necessary accretion figured on the sinking-fund basis is assumed to be paid to the owner with the annual uniform depreciation allowance, figured on the sinking-fund basis; whereas under the ordinary sinking-fund method of treatment the necessary annual contribution is made to a sinking fund which must be kept intact until the amortization of the item of property under consideration, in order that the fund may earn its own accretion. Under the former method the depreciation allowance is actually deducted from the value of the property, and the fair return is predicated on its depreciated value. Under the latter method, although the fair return is predicated nominally on the depreciated value of the property, the sinking fund must be allowed to earn its accretions, and, therefore, in effect, the fair return, until the sinking fund on any item of property is complete, must be based on the full original value, as otherwise the owner will be denied a portion of his legitimate return. An examination of the table in the Committee's report showing the annual depreciation allowances on an item of property having a life of 20 years, will make this clear. Mr. Metcalf.

Under the equal-annual-payment method of treatment, the annual depreciation allowance is increased each year over that of the previous year by the amount of interest or accretion on the payment of the last year, increasing in an item of property having a 20-year life, for instance, progressively from $3.02 per annum on the first year to $7.64 on the last year, if figured on a 5% rate, the sum of the total payments amounting to 100% in the 20-year life. This is in effect equivalent to figuring the depreciation by the sinking-fund method on the original value of the property throughout the period of life of this item, as would be done by the sinking-fund method, it being assumed, however, under the latter, that the sinking fund itself shall earn the accretion.

Why, then, it may be asked, is the equal-annual-payment method to be preferred to the ordinary sinking-fund method? The answer is to be found in the fact that the accounting is perhaps somewhat simplified, and, most important of all, that the combined payments of depreciation and fair return are, with due consideration of the increasing cost of repairs during the life of the various elements of the property, made as nearly uniform as practicable, or, at all events, are made much more uniform than by the sinking-fund method as ordinarily applied.

In its practical application, it has seemed to the writer likely to prove advantageous to establish the approximate allowances to be made for depreciation at intervals of 5 years, more or less, and to convert the estimated allowances or rates of allowance into percentages of gross revenue, rather than percentages of reproduction cost, purely on grounds of expediency and convenience in application. Under such a method it might be found, for instance, that 10% more or less per

Mr. Metcalf. annum of the gross revenue should be charged off to depreciation or amortization of property—renewals, as previously stated, being charged to new construction. Thus the depreciation allowance would grow annually with the increase in revenue, which is, of course, accompanied with increase in investment and value in property.

To attempt to apply the depreciation allowance as a percentage of reproduction cost involves keeping most careful track of the various elements of property or groups of property having lives of different estimated lengths, adding to and subtracting from these groups new plant built and old plant abandoned—a very laborious and difficult accounting problem. Under the other method of application, book accounts can be kept of the deduction on account of depreciation and memoranda of the property which has actually gone out of service. At the stated periods, then, the more difficult task of comparing the assumptions with the facts developed in the 5-year interval can be accomplished, and any necessary changes in rate allowed can be effected.

Under the equal-annual-payment method and commission control, it does not matter seriously if the allowance for depreciation is somewhat larger than necessary, provided it is not seriously burdensome in the rate, for the reason that the rate-payer immediately benefits in the reduced revenue required, resulting from the amortization of a portion of the property. The owner, on the other hand, does not suffer, as he is repaid an equivalent in cash of the property thus amortized.

Although, theoretically, if no new construction were added to the property, the depreciated value of the property would gradually decrease, as also the fair return, practically, no such condition is likely to arise, inasmuch as public service corporation properties are operated perpetually, and with the growth which is enjoyed by the majority of American cities the renewals and the new construction required will more than offset the depreciation allowance, so that the actual depreciated value of the property will practically be likely to increase rather than decrease, as will, therefore, the amount of the outstanding securities of the corporation. If, for short intervals of time, this should not be the case, then the repayment of capital through depreciation account could be carried in the bank or otherwise invested temporarily until the need does arise, or it can be used in actually reducing the mortgage debt by a retirement of a portion of the bonds, which bonds could be re-issued at a later date when the funds are required.

In the sinking-fund method of applying depreciation, the easiest practical method of accounting the sinking fund is probably the investment of the depreciation account in the bonds of the company, if the funds cannot be absorbed in new construction, and the keeping alive of these bonds, with the re-investment of the coupon interest on them in depreciation account.

Mr. Metcalf.

Mr. Alvord's comment, that if depreciation allowances are not used by the owners of public service corporations in renewals, new construction, or amortization of debt, but are used instead for outside investment, the owners must make recognition of such deduction so far as the public is concerned, is sound; and if these depreciation funds be re-invested in renewals or new construction, the writer is of the opinion that a fair rate of return should be allowed on them as allowed on the remainder of the property, and not simply bank interest rates, for the reason that in such investment in the property, hazard is assumed which must be compensated, and it is further to be borne in mind that there is always some loss in interest in the delay in prompt investment of the sinking-fund surpluses which may accumulate from time to time.

In the case of the equal-annual-payment method, it becomes a matter of indifference to the public how the depreciation funds are utilized by the owners of the property, inasmuch as the value of the property on which the fair rate of return is predicated is decreased annually by the amount of this depreciation allowance, and it seems fair and safe under operating conditions to figure the accretions of annual depreciation allowance to be paid to the owner on the basis of rates at which money could be obtained by the corporation with ample security.

Mr. Jackson.

William B. Jackson, M. Am. Soc. C. E. (by letter).—It is not possible for any one who is truly interested in this subject to read Mr. Alvord's thoughtful discussion without receiving valuable stimulus.

In the case of well-established public utility properties, it does not seem reasonable to consider that a sum of money may with propriety be laid aside in a trust fund of which the amount is determined by the amount which the reproduction cost of the physical property new exceeds the physical value depreciated, in accordance with the rules laid down by Mr. Alvord. There may be possible cases where special conditions, such as hazards of the business, might make necessary the laying aside of a sum of such amount, or even more, but the depreciation element of the property, as defined in this paper—and with which definition the writer heartily agrees—can never justify such a trust fund. As must be readily appreciated by any one who has given this matter careful study, only a relatively small part of such an amount is ever required on account of depreciation in usual public utility properties, and, such being the case, how can any one justly permit of a sum of money amounting to from 10 to 15% of the value of such a property to lie for all time in a trust fund. That is exactly what would be done if a public service company permitted a sum equal to the aforesaid difference to lie in a trust fund, and did not differentiate between the part that might be necessary for use in

Mr. Jackson. replacement and that which by the nature of things could never be thus used.

In the matter as to whether, in determining the fair value of a public utility property for rate-making purposes, the reproduction value new or the reproduction value depreciated is more important in arriving at a fair value on which the rate of return shall be made, consider for a moment a public utility property from the time of beginning business. At the beginning the company has a right to earn its current operating expenses, a reasonable sum to cover depreciation expenses, a reasonable sum for reserves, and a reasonable return on a fair valuation of its property. If the company is unable to do this during the early years of its existence, it certainly in equity has a right either to add the deficits to its investment on which it may earn a reasonable return, or to have them made up in some other way. When the time arrives when the company can begin to make money, there is no practical reason to require that it shall build up a larger reserve fund than the exigencies of the business require; therefore, it is not logical to assume that the owners have taken out a part of the value of the plant when they have not built up a depreciation fund to an amount which is entirely unnecessary for the most economical and reliable operation of the property. It is equally illogical to assume that any part of the earnings of the property should be considered as constituting such an unnecessarily large fund.

The writer believes that undue importance is placed on depreciation of plant *per se* without due consideration being given to the question whether the property has been kept in such condition as to give substantially as good and economical service as a new property and is fully safeguarded by its reserves from financial breakdown.

In this the writer does not wish to imply any lack of appreciation of the vital necessity for a vigorous carrying out of a safe depreciation programme, and his position in these matters is fully attested in his paper on this subject early in 1910 before the Western Society of Engineers, and in his discussion written in 1911 for the American Academy of Political and Social Science. There is undoubtedly no more unavoidable expense to be met by any public utility company than depreciation expense, that is, the expense for renewals of outused plant.

Mr. Lavis. F. Lavis, M. Am. Soc. C. E.—It has always seemed to the speaker to be much easier to visualize depreciation and place a value on it than to estimate appreciation properly. He is inclined to agree with the author, that it can be demonstrated, at least from a purely theoretical point of view, that when depreciation is not offset by an actual sinking fund, its value should be deducted in order to obtain the present value of physical property. When, however, valuations of railroads are made for the purposes of rate-regulation or rate-making,

it seems as if there might be good reasons for not making the deduction. Mr. Lavis.

The discussion which follows is confined to the effect of this theory on the valuation of the properties of railroads, as the speaker's experience has been principally with this section of the industries of the country coming under the general heading of public service or public utilities companies, and though it is probable that the general principles may hold good in any case, there may be some inherent differences in connection with the properties of some of the other sections, such as lighting, water supply, etc., with which he is not so familiar, which when applied to them might modify the conclusions.

It also seems best to confine any discussion of this subject to cases where the depreciation is normal, that is to say, in the case of railroads, where the plant, roadbed, equipment, etc., is maintained in such condition as to give the public reasonably full and efficient service, which with the present close inspection and regulation by the National and State authorities, should be true of most railroads. Abnormal depreciation, in any event, requires special treatment, as a reduction of rates, of course, would not tend to build up a property which is badly run down.

The theory that depreciation should not be deducted when valuations are made for the purpose of establishing rates has been advanced by C. E. Grunsky and W. J. Wilgus, Members, Am. Soc. C. E., in papers recently presented to the Society, and by H. P. Gillette, M. Am. Soc. C. E., in a brief recently submitted to the Public Service Commission of the State of Washington. Mr. Alvord also points out other differences of opinion in regard to this matter, and intimates quite truly that a proper understanding of it is not easy.

If a sinking fund is maintained to cover properly such depreciation as there may be, there can hardly be a difference of opinion that rates should be based on the full value; to this the author agrees. If, however, a sinking fund is not maintained, he claims that the value is not in the property, and, therefore, the public should not be asked to provide interest on such full value. He points out also that, in the United States, sinking funds are not usually maintained. In reference to this it may be noted that a sinking fund of 10% on the capitalized value of our railroads would amount to about $1 500 000 000 —much more capital than we can afford to have lying idle.

In order to understand the argument of those who claim that depreciation should be deducted, in cases where a sinking fund is not maintained, take a concrete example: for instance, a new railroad, the value of which, for convenience, will be assumed to be $100, and a fair return on the investment as 7 per cent. Assume, also, that the rates provide for the establishment of a sinking fund.

Mr. Lavis.

At the end of 10 years the road is rendering full and adequate service, though parts of its structure and equipment show signs of wear. The roadbed, etc., in the meantime has consolidated and its value has appreciated. Assume, however, for the purpose of the argument, that after allowing for the appreciation, etc., the net value of the depreciation is 10 per cent. The actual value of the property would then be $90 and sinking fund would be $10.

Under these conditions it will be admitted that the rates should still be maintained to provide the 7% on the amount of the full original investment, inasmuch as the sinking fund produces nothing.

During the period under consideration the annual revenue and expense account of the railroad might be approximately as follows: (Round numbers are taken for the sake of clearness.)

Operating cost	$12
Sinking Fund	1
Interest	7
Total	$20

Now, suppose that during this period the dollar, instead of being added yearly to the sinking fund, is paid out in dividends, which have been 8% instead of 7%, so that at the end of the 10 years the $10, instead of forming the sinking fund, have gone back into the pockets of the owners, then, it is claimed, the public should not be asked to pay rates on the full $100. If the case in actual practice were as simple as this, there would be little need of further argument.

The practice in the United States, however, has not been to establish sinking funds, which produce nothing, but often to put the money back into the property for betterments.

In the case of a new railroad, such as the speaker is considering, as a temporary measure, some of the slopes in the cuts are not fully taken out or not trimmed up until after it is put in operation; the line is not completely ballasted; settlement of embankments has to be taken care of, and so on. Much of this work is done during this first period with the 1% which would have gone to form the sinking fund. Later, the surplus earnings go to provide for reduction of gradients, improvements of alignment, elimination of level crossings, etc.

In Europe and in most other countries of the world, except in North America, all such work is usually charged to additional capital expenditures, and it is largely for this reason that the capitalized value per mile of railroad in Europe is so much greater than in the United States, the capitalized value of English railroads being almost twice as great. There is some idea that railroads in Europe, and particularly in England, have cost more because they are so much better; this,

however, is not so, at least not nearly to the extent of the very much greater capitalization. Mr. Lavis.

In regard to betterments and improvements paid for out of earnings, the argument, of course, is that in a properly made valuation of an existing property, the appreciation due to money spent for betterments and improvements will be duly allowed for, and that when it is, depreciation due to actual wear and tear should be deducted, in order to get the true fair value of the property in use.

If appreciation of the thousand and one improvements in the property of any well-established railroad can be properly allowed for, well and good, but, to the speaker, this seems at least somewhat difficult, whereas there is never any difficulty in allowing sufficient depreciation, so that, after all, the question resolves itself into the ability to make a true valuation of the physical property.

There is no doubt that a fair and just valuation, equitable alike to the public and to the owners, can be made, but that it can be mathematically exact is practically impossible, and because there is so much more probability of over-estimating depreciation and under-estimating appreciation, one may very well hestitate before deducting the former from a valuation to be used for the purpose of fixing or regulating rates.

In most of the valuations which have been made public, the so-called present value has been estimated to be from about 10 to 15% less than the estimated cost of reproduction new. In some cases appreciation due to the consolidation of the roadbed, etc., has been allowed, but apparently the net result is to show a present lesser value, and, according to figures generally accepted so far, the normal depreciation of a well-managed and properly maintained railroad is assumed to be about 10 per cent.

It seems to the speaker that this amount might well be, and probably generally is, offset by what might be called "not easily apparent appreciation." Any railway man of experience can easily imagine work of a physical character done on a well-established road, 20, 30, or 50 years old, which is not easily apparent, the worth of which, however, still exists.

In an article* describing the construction of a railroad recently completed near Kansas City, the Chief Engineer makes the statement that "Owing to the newness of the road, the train schedules are not yet as fast as they will be later."

In another article,† describing some recent new work on an improvement of the Nashville, Chattanooga and St. Louis Railway, occurs the following:

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* *Engineering News*, October 9th, 1913.

† *Engineering News*, October 23d, 1913.

Mr. Lavis. "On the new roadbed a heavy bed of cinders is used for ballasting, as this is more easily surfaced during the inevitable settlement of the fills, and forms eventually a good sub-ballast between the roadbed and the permanent stone ballast."

These, of course, are only specific instances of what all experienced railroad engineers know, but it seems as if the importance of this phase of the valuation question is not always fully realized, largely because many of the values thus introduced into a railroad property are not easily apparent.

There is no possible doubt that any of our modern trunk-line railroads are better transportation machines as they stand to-day, with rails and ties partly worn, with other parts of the equipment showing some wear, than they would be if we were able to replace every part in a brand-new condition. Theoretically, by taking specific instances of parts of the railroad machine, such as rails, ties, or rolling stock, one can easily visualize and value the depreciation, but it is known that the well-established railroad, as a whole, is a better and more efficient machine to-day than it was the day it was built.

Knowing this, and there can be no doubt of it, there is something wrong somewhere with a theory which gives a lessened value to a machine which is better than new, and it seems to the speaker that the trouble principally is, not the fact that depreciation is calculated and allowed for, but that appreciation is not appreciated.

In estimating the value, whether for rate-making or for any other purpose, of the purely physical elements of the property of a railroad, depreciation may be estimated and deducted, but one should also be sure that full value is given to the appreciation, to the development costs, and to the other elements which go to make up the efficient transportation machine.

Mr. Humphreys. ALEXANDER C. HUMPHREYS, M. AM. SOC. C. E.—The reading of this paper should help to emphasize a responsibility which to-day rests heavily on the engineers of the United States; namely, to educate the public and their representatives in legislatures, commissions, and Courts, many of them necessarily ignorant on the questions involved in valuations and rate-making. Here we, the engineers of the country, are charged with an especial responsibility because of our specialized training. First we should, among ourselves, do our utmost to form right judgments on these momentous questions. It is with great regret, therefore, that the speaker finds himself almost completely out of accord with the views expressed in this paper.

We are to-day passing through a period of trial for our form of government. Therefore the speaker feels particularly that, with regard to all public questions, we must frankly express our honest opinions.

This is the only way left to meet effectively the mass of bad advice which is being fed to the public. Mr. Humphreys.

In the first lines of the paper it is stated: "that space will not be taken here to review fundamentals". The author then proceeds to discuss the very fundamentals of the subject of depreciation, and to support views which, in the speaker's opinion, are fundamentally wrong.

As the speaker understands the paper, the author is attempting to confine himself practically to "the relation of depreciation to cost new", which, he says, is "vital", and that only one line of evidence (reproduction cost new) is to be considered. The speaker is not prepared to agree that "other lines of evidence are equally important before arriving at value".

Mr. Alvord then refers to some writers who have and have not agreed with him, and continues:

"Here, therefore, there seems to be a very serious disagreement, among authors who have written recently on this subject, as to whether or not depreciation should be deducted from the cost new of a property for finding value for rate-making purposes where the reproduction method is adopted as a basis of value. It seems to be desirable, therefore, to present a further study of this question."

Certainly, this paper makes it more than ever desirable that there should be further study of this question. Although the speaker is overwhelmed at the present time with duties accepted, he feels that he would be delinquent if he did not contribute his little share in this much needed discussion.

Mr. Alvord says:

"The sinking-fund method seems to be an accurate way to compute depreciation where all or most of the life history is known, as it consists largely in finding past depreciation."

First, the speaker objects to the use of the term "accurate" in connection with estimates on depreciation to accrue. Again, he objects to the intimation that, it can be assumed as a working basis that "all or most of the life history is known." This life history in large part must depend on the books of account and other records, which frequently are incomplete, and, even when complete, have been influenced by many changes in the system of accounting.

Again, in many cases, the existing company is the consolidation of many smaller companies, the books of which are often not available, and if available are found to be not self-explanatory. It would appear as if it should not be necessary to say to men of experience anything more, or even as much as the speaker has said, on this point.

The author then says:

"The sinking-fund method of computing or allowing for depreciation consists in determining the proper useful life of the structure or machine under consideration".

Mr. Humphreys. Here he appears to be confusing estimated accruing and accrued depreciation with actual depreciation; but let that be as it may, who shall say authoritatively what is the "proper useful life" of any part of a plant. There are those who assume that the life of parts of plant can be found by reference to tables prepared by so-called experts; but surely this absurdity cannot be supported by men who have had experience as constructors and operators. To determine how much should be allowed for annual expenses or cost, we may well estimate the life of each part of a plant, knowing that these figures may prove to be far from the fact; and hence we are justified in making a liberal estimate for this purpose.

Here, however, we must never forget the underlying reason for this estimate and the journal entries based thereon, namely, to spread the cost of final renewals as uniformly as possible over the periods benefited, so that we shall not deceive ourselves as to losses and consequently as to profits. Especially should we be careful not to over-estimate our profits, and hence we should be sure not to under-estimate the cost of final renewals.

The speaker wishes that the term "depreciation" had never found favor. Much of the present confusion of thought on this subject would be avoided if only the term "final renewals" were used.

Here at once it is seen that such an estimate should not form the basis for determining present depreciation. This should be determined as closely as possible by the exercise of engineering, managerial, and accountancy ability, based on academic and practical considerations, chiefly the latter.

Here we have the all-important difference between estimated accruing depreciation as an element of cost, and actual depreciation in establishing the present value of an asset; this last by actual expert examination, having in mind physical decay, obsolescence, and inadequacy.

Again, Mr. Alvord says:

"In case a portion of the life has passed away, the present value of such structure is assumed to be, in a general way, the present cost of replacement new, less the accrued amount of the depreciation fund to date".

The speaker has just shown why the amount in the sinking fund may properly be quite different from the actual depreciation. It is based on an estimate made in advance, and on many assumptions. In making an appraisal we have before us the present facts as to physical condition, obsolescence, and inadequacy. In the first case we have foresight, and in the latter we have the advantage of backsight.

To determine the actual cost, would one go to an engineer's estimate of the cost of some large construction, or to the record of the money actually expended, which would cover all omissions, contin-

gencies, overhead charges, etc.? If the first course were taken, the speaker fears that the business world would be more than ever critical of engineers' estimates, which have so often misled investors because of the omissions referred to. Mr. Humphreys.

Supposing the existence of an approximately correct determination of the actual depreciation to date, the speaker contends most emphatically that, although perhaps the plant is thus reduced in value, no deduction therefore should be made in a rate-making case.

In meeting this proposition, the question is asked frequently: "Would you pay as much for an old plant as for a new plant?" and the questioner thinks that the question settles the matter.

Not so. As a purchaser, if there were any real depreciation, the speaker would claim a reduction. Why? Because he has to assume the liability for the final renewal of the plant. In a rate-making case, however, that liability still rests on the owner, and therefore the plant must be considered as maintained or to be maintained by the owner.

Mr. Alvord, strangely enough, admits that "no deductions from cost new or reproduction cost, where that method [actual establishment of a sinking fund under trust conditions] is being used, would be necessary". Then he says:

"Where a proper depreciation fund is not set aside actually, but is only estimated and written off in the accounts, there is no such condition, and the matter of writing it off by the owner in his books amounts to only an opinion of his as to what such depreciation fund should be, but does not at all produce such a fund, or make it available; in consequence of which, an appraiser, using the cost new or reproduction method as one of the means of arriving at the value of the property, and finding no depreciation fund actually in hand or in trust, is obliged to deduct it from the property."

Is the appraiser "obliged" to deduct because of an opinion thus expressed?

Mr. Alvord shows that the owner is alive to the fact that depreciation is to be cared for, and that he refuses to inflate his profits by ignoring that fact. By his act he shows that he is liable for the cost of renewal, and that this item of cost must be obtained from income. A buyer would trade on this admission to get a lower price, but, as before shown, even then it by no means follows that the seller would have to deduct the full amount thus written off. That is not a question of the fair determination of values and the fair fixing of rates, but of bargaining.

Mr. Alvord then proceeds:

"In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand. A clear perception of this principle will save much confusion of thought."

Mr. Humphreys. Here Mr. Alvord mistakes his assumption for an established fact. The speaker fears that that is a weakness throughout the paper.

The Standard Dictionary defines "Principle", in part, as follows:

"1. A source or cause from which a thing proceeds; a power that acts continuously or uniformly; a permanent or fundamental cause that naturally or necessarily produces certain results. * * * 2. That which is inherent in anything, determining its nature; essential character; essence."

Here, then, it is found that, notwithstanding his disclaimer, as before referred to, Mr. Alvord is discussing fundamentals.

The speaker presumes that much confusion of thought might be saved if all were prepared to adopt blindly Mr. Alvord's dicta and then refuse to confuse their minds by further independent thinking.

Now, to come down to facts: Assets are widely variant in character. Cash is an excellent asset, if the money is honest money. Particularly is it valuable in emergencies because it is liquid. Usually, it is a promise to pay, on the part of the government, of a bank, or, if not in actual cash, it might be a promise to pay by the trustee of a fund; but government or bank promises to pay are not always made good. There has been no little repudiation, even in this enlightened country. Or, government notes may be greatly depreciated. We have also had that experience in the United States, a condition corrected only by the courageous act of a great President now passed to his reward. Still more, the trustee of a fund may default—not an unknown occurrence. Now, if the owner is solvent and agrees to make good the loss due to depreciation, that is, to renew the plant, and this assumed liability is shown on the books, there must be a corresponding asset, or else money not earned has been paid out in dividends. The entries referred to, however, have been made to guard against such a ruinous course. Naturally, the speaker is not referring to fraudulent bookkeeping or fraudulent management, but is following Mr. Alvord in assuming normal conditions and management.

This asset, if not in cash retained, will in all probability appear in additional plant paid for from earnings, and may be regarded as borrowed from the depreciation reserve. The fact is that where the depreciation reserve is treated in this way it has to be borne in mind that all parts of plant which appear in the appraisal inventory as having been paid for from the depreciation reserve fall in one of two classes:

1.—Parts which have been installed as renewals or replacements, and are, therefore, in place of parts represented in the original investment, and hence to be included as such;

2.—Parts of plant which have been installed as extensions or betterments, and have been paid for with money borrowed from the depreciation reserve.

These parts, therefore, should be included in the inventory and appraisal, because they are not duplications of investment, but represent additional investment. Mr. Humphreys.

In the case of an interest-bearing depreciation reserve, the balance to the reserve should be credited each year with interest at the rate agreed on. Then the money thus borrowed has to be returned when required for final renewals, and must be repaid from capital. In the meantime the company has been able to defer the day for final financing. These extensions and betterments can be considered as capital investments, the amounts borrowed from depreciation reserve standing as a liability against the owners. In reference to this matter, the point may be made again that, whether or not there is a depreciation reserve, the liability for renewals rests against the owners. The setting up of the reserve is, as before shown, to spread this item of loss as uniformly as possible over the periods benefited. The loss due to final renewals will occur in any case, though not exactly in the amount estimated.

Mr. Alvord then attempts to show that the deduction for depreciation would not lead to a progressive reduction in the rates. This discussion he carries on under the headings, "The Attempt to Simplify the Question", "Perpetual Life", "Utilities Commonly Require Growing Plants", "Earning Power of the Depreciated Plants", "Natural Accretions", and "Sinking Funds an Inherent Part of the Plant".

The speaker agrees with some of the statements he makes under these headings; but disagrees absolutely with his conclusions. On page 2053* the author states:

"It is clear, therefore, that, as far as valuation for the purpose of sale is concerned, it is right and proper that, in the absence of actual sinking funds or any sufficient reserve fund, they should be deducted from reproduction cost or 'cost new', when that method is used to ascertain value. Does not this also hold good in the case of valuation for rate-making?"

It is presumed that the author looks for a reply in the affirmative to this question. The speaker unhesitatingly replies in the negative; and this, it is thought, is where Mr. Alvord and his many adherents go astray.

The case of purchase and sale has nothing to do with the question, and the author has supplied, presumably unconsciously, many arguments to support this position. What he has to say under "Perpetual Life," can serve as an example.

What the public is concerned in is the quality of the service rendered. That is the test. It is for the commissions to see that the service is adequate in quantity and quality and that the investment and its return are not exorbitant.

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* *Proceedings*, Am. Soc. C. E., for November, 1913.

Mr. Humphreys. In purchase and sale, it is a case of bargain. It may be a case of turning a fixed asset into a liquid asset. There may be many reasons for the owner's willingness to sell well below value; but in any case he should be willing to deduct fairly for accrued actual depreciation, because the buyer has to assume the liability therefor. If the seller can turn over a sinking fund, which in the opinion of the buyer is a sufficient offset to depreciation, then his liability therefor is covered. All this provided this is the best bargain he can make; for bargain it is.

Under the heading, "The Practical Treatment of Depreciation Accounts", Mr. Alvord takes, as an example of the point he is trying to demonstrate, the case of "a railway having extra rolling stock which is only needed at certain seasons of the year when the traffic amounts to a maximum". Then he shows that the property would not be as valuable if this extra rolling stock were sold, relying on the chance of buying it back when actually needed. Again, the test is one of service. If this rolling stock had been sold before the appraisal were made, and not replaced, naturally the decrease in value would be reflected in the appraisal. If all the property had been sold, it would have a still greater effect on the appraisal, because no appraisal would be necessary.

To wind up this part of the discussion Mr. Alvord says:

"To state it in another way: a utility company plant is not as valuable to the public without a reserve fund for replacement as it is with such a fund intact and on hand and promptly available".

This is not stating it in another way, but is an entirely different proposition. The reserve fund is a matter of finance. The cars might not be purchasable for immediate delivery, but this by no means shows that money for replacements could not be obtained simply because there was not cash enough on hand. Such a case is almost unbelievable, provided the owner enjoyed the ordinary good credit of public service corporations. Again, it is a question of service, and the public is not interested in the financing of the replacements provided the amounts thus expended do not add to the capital burden.

Under the head of "Outside Investment of Depreciation Funds", Mr. Alvord objects to investing the depreciation reserve in betterments which might give 7 or 8%, instead of the guaranteed $3\frac{1}{2}$ to 4 per cent. He refers to "the general risk incident to the enterprise". As all the capital invested is subject to this risk, there is no good in splitting hairs about this particular part of the investment; but Mr. Alvord thinks that it works an injustice to the public if the owner gets a return on these betterments. If the return has been allowed for in the amount annually to be set aside, it does not. The returns are in lieu of the interest otherwise allowed by the banker who holds the depreciation fund, and these returns are required to bring the reserve up to the required amount. As to the extra rate of earning, there

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can be no injury to the public if the property prospers because of this or any other saving. Also, the speaker imagines that Mr. Alvord has failed to appreciate the fact that, as a rule, extensions thus made do not pay a high return at the start and are frequently made in advance of a paying demand for the service.

The speaker, as a manager, has had to face this problem in not less than fifty cities in the United States.

Under the head of "Present Practice" Mr. Alvord says:

"The rate commissions have not yet discussed this question [that of deducting a 'theoretical depreciation fund'], nor has it been effectively presented to them, but it is believed that, when discussed, it will be along lines which are here indicated."

Believed by whom? The speaker quite disagrees as to the correctness of the statement. He can hardly think of any question which has been more discussed in the cases of which he knows. He also ventures to believe that the questions involved have been "effectively presented". The statements have not always been understood, for those to whom the statements and arguments were addressed were in some cases, through lack of training and lack of experience in the several lines of construction and administration, incapable of understanding the matter presented.

Under the head of "Should a Sinking Fund Earn Full Return", Mr. Alvord says: "We will have to await with patience the conclusions of the older commissions which are giving this matter study". Again, the speaker completely disagrees with him. It is not for us to "await with patience", or with impatience, the conclusions of the older commissions, or the younger commissions, on any point. It is for us to go ahead with open minds to determine what we believe is fair and honest to the public and public utilities, and to do our best to educate the public and the commissions. If they are right and we are wrong, it is for us to acknowledge the fact; it is for them to do the same. It by no means follows that we are to accept as correct the rulings of the commissions simply because we have to govern ourselves thereby until these decisions are reversed. Even the Supreme Court of the United States is not infallible.

The speaker also objects to the reference to the "older commissions", as if they of necessity were most likely to give just decisions. That does not follow. There may be an old commission with a new personnel, and "the last state may be worse than the first".

Under the head of "Depreciation Accounts", Mr. Alvord indicates that in his opinion the cost of depreciation should be estimated liberally. He says, "Some allowance must be made for the unknown". Also, "From time to time adjustments will have to be made to correct for errors in judgment as to future life". This is quite true; but the unknown has not happened when the rate-making appraisal is

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made, and if no "adjustments * * * to correct for errors in judgment" have been made, how about deducting from cost new the full amount thus accumulated in the reserve?

In the speaker's experience the so-called experts testifying for the people deduct the depreciation according to an assumed "average life", and they do not worry about the "unknown" or "errors in judgment". They are so used to dealing both in the unknown and errors in judgment that they have long since ceased to worry.

On page 2060* Mr. Alvord says:

"When renewals are made, the depreciation account should be debited with the outside capital replaced in the plant by the owner, and credited by the cost of the improvement."

Is this one transaction which is referred to? Is it a replacement? If so, is it an improvement? Of course, a replacement may include in part the element of improvement, but this hardly seems to be what is intended. To what account is "the cost of the improvement" to be credited?

The speaker does not find this a self-explanatory direction for a journal entry, and would suggest that it be rewritten.

Now, as to the several items of the summary:

1.—This is incomplete.

2.—The speaker disagrees with this, absolutely.

3.—"All the facts of the past are" not known. The sinking fund is not the most accurate method of computing depreciation. The depreciation has to be estimated before the sinking fund can be computed.

4.—This is quite correct, and furnishes an answer and contradiction to much that is in the paper.

5.—The reference to "private gain" is not clear. If the speaker understands the statement, he considers it an unfair proposition.

In his concluding words, the author says: "all the principles which it is endeavored herein to reason out are not yet generally accepted".

The speaker is glad to be able to agree with Mr. Alvord here; but must go farther and express the hope that most of these "principles" never will be "generally accepted". They cannot be accepted without working confiscation in some degree.

In conclusion the speaker begs to refer to a paper recently written by him which discusses at length most of the questions raised by Mr. Alvord. This paper is entitled "Depreciation, Estimated and Actual." It was presented in June, 1913, before the Institution of Gas Engineers of Great Britain, and has been reprinted in many of the technical journals of the United States, and also in pamphlet form.

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* *Proceedings*, Am. Soc. C. E., for November, 1913.

In this paper the speaker supports the proposition that no deduction should be made for depreciation if the property has been adequately maintained, and particularly that no deduction should be made on account of aging of the plant. He draws the line sharply between estimated accruing depreciation and actual present depreciation, the latter as found by expert examination at the time. He also shows that in not a few cases there is no need for any estimated depreciation because the current final renewals make a fairly uniform annual charge, and this applies to properties as a whole, and in many more cases to parts of properties, so that those parts can be eliminated from the estimate of accruing depreciation. Mr. Humphreys.

Henry Floy, Esq.—Without discussing in detail the points made by Mr. Alvord, some of which are well taken, the paper, as a whole, is wrong in its conclusion and unfair both to the utilities and the public. Mr. Floy.

The author bases his paper on two assumptions—which he does not attempt to prove—both of which are false:

First.—That both newly originated and long established utilities must be treated alike.

Second.—That a depreciation reserve fund is a necessity.

Before undertaking an intelligent discussion of this subject, there must be drawn a sharp line of demarcation as to whether the valuation being considered is that relating to an existing and long established corporation, or one which has originated under public service regulation. The same rules may not fairly be applied equally to utility corporations so differently originated.

In a case of property constructed and put into operation before the creation of regulating commissions, the investors, with the consent of municipal, State, or Federal authorities and the sanction of the public at large, were allowed to do certain things which are no longer permitted under utility regulation. Where a corporation is created and the money of the owners is invested, after notice, under conditions prescribed in advance by regulating bodies, there can be no fair or honest objection to the insistence on accumulation of depreciation funds out of revenue and the use of such funds in any reasonable way, but they cannot necessarily be used as a measure of the value of the property on which rates are to be fixed. With respect to corporations that have been in existence for a considerable period of time without commission regulation, and have been following certain generally accepted principles with the consent, more or less definite, of public authorities, the speaker takes emphatic exception to the proposition advanced by the author, that value for rate-fixing purposes must be determined by deducting the amount of a computed but not accumulated reserve fund from the cost of reproduction new. The arguments presented in the paper may be reasonable when applied to some

Mr. Floy. newly organized corporations, but they are unfair as applied to most existing corporations. The point to be made clear is that no *ex-post-facto* rulings should be attempted by commissions or countenanced by such organizations as the American Society of Civil Engineers, simply because no reserve fund is in hand. Who shall say that, because utilities had not accumulated reserve funds before public regulation was instituted, therefore, the corporations shall now be deprived of a part of their property. In determining rates where the public and corporations have consented to the existence of a certain thing in the past that is not legally or criminally wrong, and about which there still exists honest differences of opinion, it is not fair or right, in making rulings for the future, to attempt to apply such rulings to the past at the sacrifice of investment or the jeopardy of income. It is on this point—the difference between a corporation already in existence and one originated under new laws—that the mistakes have been made in the rulings of commissioners who, for the most part, inexperienced in utility corporation affairs, have attempted to apply theoretically perfected methods which are largely the result of development and experience. It is the old truth, that the same medicine will make some men well and others ill.

That the so-called, though misnamed, present value is not determined by the cost new less the amount of depreciation or reserve fund is indicated by the fact that a fund accumulating for a period of years, based on the original investment, would probably result in too small deduction from the reproduction cost new in case of investments which have enormously increased in value, and too great deduction in the case of those which have largely decreased in value.

In the second instance it may certainly be questioned whether there is any necessity for the accumulation of a reserve fund for the purpose of ultimately returning to the investor full value of his property, except in a case of limited franchises or where the lives of utilities are definitely determined. Although in most cases a moderate reserve fund should be kept in hand for contingencies, it is a fact, as has been argued for some years by the speaker, that there are utilities which do not require the accumulation of any depreciation reserve funds whatsoever. A corporation which has its property widely distributed, of various characters, with single units of relatively small value, may not require any depreciation reserve because the deterioration of property—after the first few years of operation—is taking place at an approximately uniform rate per annum, and, therefore, all depreciation becomes wear and tear and is taken care of as a part of the annual operating expense; as an example the Third Avenue Street Railway Company, of New York City, may be cited. This property consists of nearly 300 miles of surface track, partly overhead and partly underground trolley, with rolling stock to correspond of all

sizes and types, as well as storage battery cars, underground conduits, cables, overhead transmission systems, car barns, sub-stations, generating station, real estate, and other property extending from the lower end of the Island of Manhattan up to and through the Bronx and the County of Westchester, so that the exhaustion of any physical property, as far as can be anticipated, will not result in unduly increasing operating expenses for its replacement, or affecting net income, or in any way jeopardizing service or charges to the public. Despite the orders of the Public Service Commission to do so, the Third Avenue Company accumulates no depreciation fund, and yet is operating most successfully. Mr. Floy.

Why, then, should Mr. Alvord argue that this company should have on hand a reserve fund equal to 15 or 20% net value of its property, say, from $9 000 000 to $15 000 000, for which the corporation has no use, except to be allowed to earn fair return on this property under his theory.

On page 2055,* Mr. Alvord says, "a public utility owner, under such circumstances, is tempted to let his plant run down, and withdraw from the property the funds necessary for its renewal from time to time, and use such funds for his other gainful purposes." Except under abnormal conditions, this statement is not in accordance with the facts. Valuations of a large number of utility properties of different kinds, made by a host of independent and separate appraisers, result in showing that the average condition of these properties is between 80 and 90% of reproduction value after deducting the amount of depreciation, as determined by methods suggested by Mr. Alvord, or the straight-line method of calculating depreciation which shows even a smaller present value than Mr. Alvord's method. Consequently, it may be stated, without fear of contradiction, that the condition of the average utility plant is not run down, whether or not reserve funds are maintained. Moreover, utility regulation provides for proper maintenance and service, and this can be enforced and secured even to the exclusion of dividends. Take the case of a corporation which has issued only stock, and has no bonds or mortgage indebtedness outstanding—as is not unusual, particularly in New England—in such a case, the entire interest of the stockholders is a guaranty of good service. Why, then, must the public be further protected by an unwieldy and useless reserve fund, which is of no service to the corporation, except to permit it to earn on the full value of the property?

CLINTON S. BURNS, M. AM. SOC. C. E. (by letter).—The question as to whether or not depreciation should be deducted from the cost new of a property in finding the value for rate-making purposes where the reproduction method is adopted as a basis of value, is one that has Mr. Burns.

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* *Proceedings*, Am. Soc. C. E., November, 1913.

Mr. Burns. often suggested itself to the writer, but not until recently has he undertaken to make a comprehensive analysis of the subject with regard to the equity of the points involved. Considering the case of the most simple proposition, as cited by the author, where the composite life of an entire plant has been estimated, at the end of which the plant would be subject to entire renewal; that is to say, the original investment will have completely disappeared at the end of this estimated life, the following analysis suggests itself:

First, let it be understood that the obligation to renew the property, or any portion thereof, is a liability of the public utility company, and if the company has not the cash in hand with which to meet this liability, it must assume the burden of financing the necessary renewals whenever the necessity may arise, and as far as the public is concerned, it is immaterial whether it chooses to provide in advance for the financing of renewals by setting aside its annual accretions to the depreciation fund, or, whether it chooses to make other uses of these funds and then re-finance the renewals when the necessity arises—provided, however, that it is strong enough to accomplish such re-financing promptly when called on to do so. Furthermore, it is now conceded by all authorities that the obligation to renew the property, or, in other words, to make good the depreciation, is very properly transferred to the public (the rate-payers). This is expressed in substance by the Supreme Court of the United States in the Knoxville case in the following words:

"The Company is not bound to see its property gradually waste without making provision out of the earnings for its replacement. It is entitled to see that from the earnings the value of the property involved is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning."

This language clearly defines the underlying principles which should govern an analysis of depreciation. Depreciation is an obligation to be met by the rate-payers—the public—at the time when renewals become necessary. Stating the problem in its simplest form, freed from all legal entanglements, and considering only the equity in the case, the following analysis results:

An investor, on entering the public utility business, dedicates his property to the public use, and, in return, the public obligates itself to pay a reasonable rate of return for the use of the property and to restore to the owner the value of any portion of it consumed or wasted in the public service; or, in other words, the public is obligated to make good the depreciation. Suppose, now, that the utility, at the beginning of the enterprise, should accept from the public the promissory notes of all the rate-payers, payable on demand, without interest, the face value of these notes aggregating an amount equal to the reproduction cost of the property, it being stipulated that the payments

would be demanded only as required for the renewal of totally depreciated property. If such a procedure were practical, and assuming that these notes were convertible into gold dollars on demand, at full face value, depreciation would thus be provided for; and an appraiser, called on to determine the value of the property for rate purposes, would undoubtedly recognize the injustice of deducting depreciation from the cost of reproduction in any project financed in this manner. Now, the foregoing procedure is exactly what the sinking-fund theory is, except that the investor allows these promissory notes to be paid in small annual payments, just sufficient so that if invested at compound interest, the total sum accumulated at the maturity of the notes (the expectancy of the property life), will equal their face value. The investor, therefore, in accepting depreciation on the sinking-fund basis, does not in fact, so far as its use value is concerned, have any of his available capital returned to him until the end of the life of the structure, and in order that he may receive a fair return on the value of his property, the rates must be based on the reproduction cost of the property, undiminished by depreciation. If the investor chooses to reinvest the cash on hand in the depreciation fund at any time, with the hope of securing greater returns therefor, he must assume the risk and incur the liability of returning the amount withdrawn plus its accumulations at compound interest when necessary for renewal expenditures; for example, if he is collecting depreciation on the sinking-fund basis, where 4% is considered a fair rate of interest at which to compound the fund, and he is able to withdraw the fund and invest it in extensions to the property, or elsewhere, and thus realize 8% thereon, the additional 4% is due him as compensation for the effort, risk, and worry inherent to the investment of capital, and the first 4%, being the proper rate for liquid capital, must be considered as going to restore depreciation. Mr. Burns.

The writer fails to see why any distinction should be made between the case where the sinking fund is actually kept in the bank or in trust and where it is kept merely as a book entry and has been diverted to other investments. In fact, the public, by the fundamental principles on which the sinking-fund theory is based, compels the utility owner to invest the depreciation fund somewhere, where it will draw 4% compound interest, or some predetermined rate considered proper for liquid capital; and how can it matter to the public whether it is invested in time deposits in a bank, in Government bonds drawing the necessary rate of interest, or in private venture where the owner perhaps hopes to realize 10%—and what matters it whether or not his expectations are realized? The public secures its recompense in being permitted to discharge its depreciation obligation by paying into the sinking fund the smaller annual contributions and imposing on the owner the obligation of reinvesting the fund; and the owner secures

Mr. Burns. his reward for reinvestment if this venture proves successful, and he pays the penalty if it is a losing one.

The author states further "that the owner cannot have a return on it [the sinking fund] as a 'sinking fund' for depreciation, * * *, and at the same time have a return on it as a betterment to the plant. This would be an obvious injustice to the public." At first thought this seems plausible, but, on further consideration, is it not a fact that the contrary is the case, namely, that the public in reality owes this opportunity for reinvestment to the utility owner?

The public, having imposed on the public utility owner the necessity for reinvestment of the fund where it will earn, say, 4% interest, should in equity open the avenue of investment to him when the opportunity presents itself. To illustrate further, take the simplest case imaginable; for instance, a gravity system of water-works, where the entire property consists of $100 000 worth of wood pipe, having a life of 20 years; the utility owner dedicates this $100 000 to the public service, and, in return, the rate-payers obligate themselves to pay a reasonable return on the investment, and to preserve the property intact; or, in other words, make good the waste from usage. The rate-payers provide for the depreciation by giving their promissory notes aggregating $100 000 in face value, payable 20 years after date, without interest. The utility owner permits these notes to be paid in annual installments on a 4% sinking-fund basis, payable at the beginning of each year, or at the rate of $3 229 per annum; and it so happens that the betterments, which in this instance are for extensions to the pipe system, require the expenditure of $3 229 per year. The rate-payers, in consideration for being allowed to care for depreciation by paying the sum of $3 229 annually, permit, and, in fact, compel, the utility owner to reinvest this sum of $3 229 annually where it will earn 4% compound interest, and they also require him to invest this or some other $3 229 annually in pipe lines for extending the service to the new territory; but they pay him extra in consideration for these new extensions an amount equal to 3.229% to restore depreciation and, in addition thereto, a reasonable return on these further sums invested annually.

Two courses are open to the investor: he may leave the first $3 229 annually in a trust fund, earning 4%, in some reliable bank, and use $3 229 annually of his other resources in extensions, or, he may expend the $3 229 depreciation fund directly for extensions. In the former case he will find his situation in 20 years to be as follows:

He will have $100 000 in the trust fund with which to replace the original pipe system, and he is also in possession of $100 000 worth of extensions paid for from his outside resources. In the latter case, he will find at the end of 20 years that he has $100 000 worth of pipe requiring replacement, with no funds in trust, but he has the $100 000

in his outside resources which he may as well now spend all at once instead of at the rate of $3 229 annually, as was done in the former case. He also has the same investment in extensions or betterments as in the former case. Hence, the owner's financial situation is exactly the same at the end of 20 years regardless of which method of finance he chooses to pursue. The public is likewise in identically the same position in either case; for, had the owner left the depreciation account in trust until the $100 000 had accumulated, at the end of 20 years that sum is due the owner in exchange for his worn-out property, dedicated and consumed in the public service. Hence, the public could have no claim on the sum, nor title to any portion of it. Neither the owner of the utility nor the public being in any wise affected by the owner's withdrawal of the sinking fund for reinvestment or for extensions of the property, it would seem proper that the valuation for rate purposes should be the same in either case, no matter whether the sinking fund is left in a trust fund or used for private venture. If the amount of depreciation to be set aside annually is computed on the sinking-fund theory, it, therefore, seems that no deduction can equitably be made from reproduction cost in arriving at the proper valuation to use as the basis of rates. The rate-payers, therefore, should contribute annually $3 229 for depreciation and the reasonable rate for interest and profits to the owner, regardless of whether or not the depreciation fund is held in reserve. Mr. Burns.

The writer, therefore, can see no reason in equity for the conclusion that any injustice to the public results from allowing the owner of the utility to have return on the sinking fund for depreciation at the proper rate of interest for such depreciation accounts, and at the same time have return on it as betterment to the plant, for each betterment to the plant simply means a larger investment or larger plant on which the owner is entitled to the proper reasonable rate of return. Doubtless what the author had in mind was that it would be unjust to allow the owner to reinvest the sinking fund in betterments and receive thereon, say, 8% net return, and, at the same time, receive 4% as sinking-fund accumulation. It seems to the writer, however, that when the owner invests his sinking fund in betterments, or in private enterprise, and thereby receives 8% return, he in fact, does not receive 8%, but, on the contrary, receives only 4%, because half of this 8% return does not represent profit but simply goes to restore to him some of his property already consumed by the public in the public service.

If a community constructs a utility under municipal ownership, it is almost universally the custom to pay for such a utility by a bond issue. Now, to determine the basis of rates in such a municipal enterprise, in order that it shall be made exactly self-supporting, with no surplus, the amount of the original bond issue is always taken as

Mr. Burns. the basis on which the community must pay interest, assuming that the sinking fund with which to pay off the bonds is based on an expectancy equal to the life of the property; that is to say, if the bond issue runs for the number of years represented by the life of the property, the community must set aside a sinking fund sufficient to pay off the bonded indebtedness, and must likewise continue to pay interest on the full amount of the bond issue, regardless of the fact that the property itself is continually depreciating in value. In fact, this method of financing the enterprise is absolutely essential if the project is to be made self-sustaining. How, then, can there be any escape from the conclusion that a privately owned utility must receive its return on the same basis, for to receive less will undermine the integrity of the investment, and is, therefore, not a fair return. Of course, if the municipal bond issue is to be paid serially, then the amounts paid off from time to time would be deducted from the capital on which the public must continue to pay interest, but in order to pay these bonds serially, the annual levy or contribution for sinking-fund purposes must be largely in excess of the amount necessary for the application of the sinking-fund theory. It would seem, therefore, that the proper deduction to make from reproduction cost is simply the amount by which the actual depreciation allowance exceeds the theoretical accumulation in a sinking-fund account, computed on the age and life of the property. If depreciation is figured on the sinking-fund basis only, therefore, no deduction should be made from reproduction cost.

The Public Utilities Act of California, in effect March 23d, 1913, provides, in Section 49, that:

"The Commission shall have power, after hearing, to require any or all public utilities to carry a proper and adequate depreciation account in accordance with such rules, regulations and forms of account as the Commission may prescribe. The Commission may, from time to time, ascertain and determine and by order fix the proper and adequate rates of depreciation of the several classes of property of each public utility. Each public utility shall conform its depreciation accounts to the rates so ascertained, determined and fixed, and shall set aside the moneys so provided for out of earnings and carry the same in a depreciation fund and expend such fund only for such purposes and under such rules and regulations, both as to original expenditure and subsequent replacement as the Commission may prescribe. The income from the investments of moneys in such fund shall likewise be carried in such fund."

With the administration of the depreciation fund in accordance with this Section of the California Act, it would seem that such fund must be kept separate and set aside, and as far as the owner of the utility is concerned, he has no discretion whatever over its disbursement, and he cannot withdraw such fund for any purpose other than for the replacement of depreciated property. Therefore, depreciation

cannot equitably be deducted from the cost of reproduction in California rate cases when that method is used to determine the value of the property. Mr. Burns.

Section 47 of the Public Utilities Act of Idaho, and also Section 79 of the Public Utilities Act of Missouri, use almost exactly the same language as the California Section already quoted. This indicates the tendency in recent legislation to require public utility companies to set aside sinking-fund accounts, or, in other words, it takes these accounts out of the hands of the public utility owners and leaves them largely under the control of the public through its representatives, the public service commissions. There can be no objection to leaving the collection and disbursement of the sinking fund entirely under public control through proper utility commissions, and, in fact, it is highly desirable that such should be the case, but it is important that the commission should study carefully the conditions surrounding each property in order that the amount of depreciation collected from the rate-payers, and thus set aside, shall be adequate and equitable under all the circumstances.

Contingent depreciation should receive special consideration in each case, its amount depending on the nature of the property. Contingent depreciation is the sum that should be provided from earnings to meet unusual or unexpected replacements which become necessary from time to time in the experience of every public utility, such, for example, as damage from floods, destruction of property by earthquake or ice gorge, failure of water supply, and many similar experiences. Depreciation by obsolescence or changes in the art should also be fully covered by the allowance for contingent depreciation. In fact, contingent depreciation may be defined as the sum that should be provided for insurance against such contingencies as cannot be covered by any class of commercial insurance. This raises an important question: Is the sinking-fund method of computing the depreciation fair to the owner of a utility in a rate-regulation investigation? Evidently not, unless the rates are based on the cost of reproduction undiminished by depreciation, but, under the usual procedure in a rate investigation, it is extremely difficult, if not utterly impossible, to receive consideration for the full reproduction value of a public utility, and would it not be more practical, therefore, to adopt some system of accounting depreciation that will secure equity to both parties when considered in conjunction with a proper rate of return on the remaining expectancy of the property? Perhaps this can be understood more clearly by considering the property as passing through a series of successive ownerships. It cannot be denied that the owner may sell the property at any time and retain the accumulated depreciation fund. This fund is due him, for, if he sells the property at its fair value, computed

Mr. Burns. on the sinking-fund basis, he must retain the depreciation fund in order to recover the full amount of his investment.

Consider again the case in its simplest form, the gravity system of water-works previously cited, and, for the purpose of simplicity, assume that no extensions or betterments are required throughout the 20 years' life of the property, and that no elements of appreciation in value accrue to affect any portion of the works. The first owner should receive from the rate-payers the sum of $3 229 for depreciation and a reasonable return on his investment of $100 000. He sells the property at the end of the first year at its value, computed on the sinking-fund basis, namely, $96 771. The second owner should receive from the rate-payers for depreciation the sum of 3.475% of his investment of $96 771, or $3 362.79—this being computed on the basis of the remaining life of the property, 19 years—and, in addition thereto, he should receive a reasonable rate of return on the reduced capital account, $96 771. Now, if such a rate is equitable to each of the successive owners of the property, it must be equally fair to the public and to the owner throughout the life of the property, if the same ownership continues throughout the entire period. For the purpose of illustration, Table 3 is submitted, computed on the basis of 4% compound interest on the depreciation fund, 8% for interest and profit on the capital investment, and a variable allowance for contingent depreciation, starting at 2% of the capital account and increasing as the property becomes older, the amount of increase being such as to preserve a constant total contribution from the rate-payers. It is desirable, from the point of public expediency, that rates be not fluctuating, and this may be accomplished by the variable contingent depreciation, based on an amount that shall represent, as nearly as can be predetermined, the facts in each case under consideration.

An examination of the column in Table 3 headed "Contingent Depreciation", shows that the variation in contingent depreciation from year to year is not great, under this system of accounting, and represents very closely what may be reasonably expected, increasing gradually with the age of the property. The contingent depreciation is not a cumulative fund, but, on the contrary, represents the amount actually spent from time to time in replacing obsolete or abandoned property and in repairing damage from unexpected or unusual causes. If properly proportioned, such a fund will fluctuate between surplus and deficit, but will maintain a general average near the zero line. The actual figures used in Table 3 are, perhaps, not appropriate for a property of this nature, but are submitted simply to illustrate a principle and without reference to the adequacy, or otherwise, as this can be determined only by a study of all the circumstances surrounding each particular property. The table is based on annual contributions to a sinking fund at the beginning of each year, although many

authorities prefer using the tables computed for the end of the year. Rates are usually collected quarterly or monthly and, therefore, as a matter of fact, it would more nearly accord with correct theory to use tables computed for annual contributions in the mid-year, if such tables were at hand. However, this difference is not sufficient to cause material error in any event. Mr. Burns.

TABLE 3.

Owner.	Capital invested.	Depreciation.		Interest and profit. 8 %.	Contingent depreciation.	Annual cost to rate-payers.
		Rate. Per cent.	Amount.			
1st.........	$100 000	3.229	$3 229	$8 000	$2 000	$13 229
2d.........	96 771	3.475	3 363	7 742	2 124	13 229
3d.........	93 408	3.749	3 502	7 473	2 254	13 229
4th.........	89 906	4.058	3 648	7 193	2 388	13 229
5th.........	86 258	4.406	3 801	6 901	2 527	13 229
6th.........	82 457	4.802	3 960	6 597	2 672	13 229
7th.........	78 497	5.257	4 127	6 280	2 822	13 229
8th.........	74 370	5.783	4 301	5 950	2 978	13 229
9th.........	70 069	6.399	4 484	5 606	3 139	13 229
10th.........	65 585	7.130	4 676	5 247	3 306	13 229
11th.........	60 909	8.009	4 878	4 873	3 478	13 229
12th.........	56 031	9.086	5 091	4 482	3 656	13 229
13th.........	50 940	10.435	5 316	4 075	3 838	13 229
14th.........	45 264	12.174	5 554	3 621	4 054	13 229
15th.........	40 070	14.496	5 809	3 206	4 214	13 229
16th.........	34 261	17.753	6 082	2 741	4 406	13 229
17th.........	28 179	22.643	6 381	2 254	4 594	13 229
18th.........	21 798	30.803	6 714	1 744	4 771	13 229
19th.........	15 084	47.134	7 110	1 207	4 912	13 229
20th.........	7 974	96.154	7 667	638	4 924	13 229

Unfortunately, in actual practice, the problem is never so simple as the gravity water-works system used in this illustration. Public utility enterprises are, from the very necessity of the case, almost continuously extended from year to year, so that the average age of a property of this nature never even remotely approaches that of its oldest parts.

If the theory outlined herein is accepted as a correct method of computing depreciation, its practical application would be somewhat as follows:

The appraiser would determine the composite life of the utility under consideration, which, for illustration, in the case of a water-works system, in some particular case might be found to be 60 years. The average age of the entire property, computed in like manner, might be found, for example, to be 20 years. The rate to be used in determining the amount of ordinary depreciation therefor would be based on an expectancy of 40 years. On a 3% sinking-fund basis, this would be 1.288%—this percentage to be applied to the remaining capital determined by deducting the sinking-fund accumulation of 20 years from the cost of reproduction of the property. The proper

Mr. Burns. additional amount to be allowed for contingent depreciation, of course, should be determined by the particular circumstances in the case. It would be more precise to compute the result separately for each group of items having the same age and estimated life; but such a procedure would, perhaps, be impractical in most cases, owing to its difficulty of application.

Mr. Knox. STUART K. KNOX, M. AM. SOC. C. E. (by letter).—Mr. Alvord's paper, dealing with "the general relation of depreciation and its effects on the fair return of a utility property," will have justified itself if it does no more than precipitate the extended discussion of this interesting and intricate subject, concerning which ideas are at present so confused and conflicting. In this, it appears to the writer, will lie its chief, if not its only, value.

As the author implies, there exists at the present time a wide diversity of opinion among engineers, lawyers, special masters, commissions, and Courts having to do with valuations for rate-making purposes, with regard to the fundamental question of the value on which fair rates should be predicated.

Among those who have sought to sustain the reproduction-cost-new theory may be mentioned the special master,* *re* Columbus Railway and Light Company *vs.* City of Columbus, Ohio, 1906; the Wisconsin Railroad Commission,† *re* City of Whitewater *v.* Whitewater Electric Light Company, decided December 16th, 1910; Massachusetts Joint Board,‡ Massachusetts Appraisal of the New York, New Haven, and Hartford Railroad, 1911; C. E. Grunsky,§ M. Am. Soc. C. E., in his paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates", to which paper the author refers; and Alexander C. Humphreys, M. Am. Soc. C. E.,|| President of the Stevens Institute of Technology, in his paper entitled "Depreciation: Estimated and Actual", presented before the Institution of Gas Engineers of Great Britain, 1913.

The contrary view, to the effect that fair rates should be adjusted to yield a fair return only on depreciated reproduction-cost, has been upheld by the Oklahoma Supreme Court,¶ *re* Pioneer Telephone and Telegraph Company *v.* Westenhaver, decided January 10th, 1911; by the New York Public Service Commission for the First District,** *re* Metropolitan Street Railway Reorganization, decided February 27th, 1912; and by the United States Supreme Court.

Mr. Alvord's ideas on the subject are summarized in his first and second "conclusions," as follows:

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* Robert H. Whitten, "Valuation of Public Service Corporations," p. 368.
† *Loc. cit.*, p. 366.
‡ *Loc. cit.*, p. 363.
§ *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.
|| *Transactions*, Institution of Gas Engineers (Great Britain), 1913, p. 348.
¶ Robert H. Whitten, "Valuation of Public Service Corporations," p. 375.
** *Loc. cit.*, p. 376.

Mr. Knox.

"1st.—That if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property 'used and useful for the public'.

"2d.—That if these funds are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope, as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain."

The writer is not prepared to admit the truth of either of these propositions without material qualification, and in the following discussion will seek to show:

(*a*) That the disposition made of the depreciation reserves has theoretically no bearing whatever on the question of value on which the owner of a utility property is entitled to earn a fair return.

(*b*) That the question of value on which an owner is entitled to earn a fair return depends solely on whether the depreciation reserve provided has been or is to be just "adequate", "excessive" in amount, or "inadequate" for the purpose intended.

He will endeavor to demonstrate that when the depreciation reserves, which have been and are to be provided, are just "adequate" for the purpose intended, the owner must, if he is to be protected against the impairment of his capital or of its earning power, be permitted to earn, in addition to operating expenses and this "adequate" depreciation reserve, a fair return on reproduction-cost-new.

That when the depreciation reserves, which are to be provided, are "excessive", so as virtually to effect a refund to the owner of a portion of his original investment, together with its full earning power, rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new, the measure of the reduction being the total amount of the capital so refunded.

That when the chosen method of providing for future depreciation is such as virtually to effect a progressive refund, free and clear, of a portion of the owner's original investment, a corresponding progressive decrease in the value on which he is allowed to earn a fair return may be equitably made.

That when the depreciation reserves which have been provided are "excessive," so as virtually to have effected a refund of a portion of the owner's original capital, rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new only if no change in ownership has taken place.

That when the depreciation reserves which have been provided are "inadequate", so that an actual impairment in capital has taken place.

Mr. Knox. we should, to be consistent, and provided full ownership is still retained by the owner who suffered the loss, permit the adjustment of rates to yield a fair return on a value greater than reproduction-cost-new until this loss has been recouped.

That rates may be equitably adjusted to yield a fair return on depreciated reproduction cost, only if the virtual refund of capital, resulting from "excessive" past or prospective depreciation reserves, just equals the accrued depreciation on the property on the date of the valuation for rate-making purposes.

As a preliminary to the intelligent discussion of the preceding propositions, it is essential first to define exactly what is meant by "adequate", "excessive", and "inadequate" depreciation reserves, and by the expressions "accrued depreciation" and "depreciated reproduction cost".

By "adequate" depreciation reserve is meant the annuity or percentage of reproduction-cost-new which, if set aside in the bank or in trust from the beginning of operation, at the rate of interest, say 4%, on which the fund payments are predicated, will, with all interest accretions, amortize a sum which, at the termination of the useful life of each element of the property, will just suffice to replace it and no more.

By "excessive" depreciation reserve the writer means an annuity greater in amount, and by "inadequate" depreciation reserve an annuity less in amount than the above.

By "accrued depreciation" at any period of the life of a property, is meant the amount which an "adequate" depreciation reserve would have amortized had it been set aside each year from the beginning of operation.

By "depreciated reproduction cost", is meant the reproduction-cost-new less the "accrued depreciation" so determined.

By the "useful life" of any element of property, is meant its age plus the remaining life expectancy of the element.

In explanation and justification of the foregoing definitions, it may be well to digress at this point to the extent of acknowledging a full realization of the fact that any estimate of accrued or prospective depreciation is at best a guess. There are, however, intelligent guesses and unintelligent guesses, and there are intelligent ways of guessing and unintelligent ways of guessing. One unintelligent way of guessing the accrued depreciation on a pumping engine, for example, is to sit in the office and compute it, starting with the assumption that the normal life expectancy of a new unit of the type considered is 20 years, and that the age of the particular unit under consideration is, say, 10 years. The unit may have been in continuous service from the date of its installation, or it may have been used largely as a spare. It may have been operated 24 hours per day, or only 1 or 2 hours per

day. It may or may not have had intelligent care, and it may or may not be well adapted to the service, well designed, and of modern type. No intelligent guess of accrued or prospective depreciation can be made without a visual field inspection of the element to be appraised. Mr. Knox.

Whatever be the age of a unit, or structure, the present value of which it is desired to ascertain, the uncertain element affecting this value which may be guessed most accurately by an expert is its remaining life expectancy. In the light of the history and probable future of the plant, of the community which it serves, and of the arts, this, after an examination of the unit or structure, may be gauged as closely as any other probability affecting the future of the property. Then, if the guess proves accurate, the "useful life" of the element becomes its age plus its life expectancy on the date of the appraisal; and an "adequate" allowance to cover annual depreciation is the annuity, which, had it been paid into a fund each year since the installation or construction of the unit or structure, would have amortized a sum sufficient to reproduce it at the end of this "useful life". Under these conditions the "accrued depreciation" on the date of the appraisal may fairly be estimated as the amount which such an annuity would have accumulated between the date of installation and the date of the appraisal, and its fair market value or depreciated reproduction-cost at the date of appraisal may fairly be assumed to be its reproduction-cost-new less the accrued depreciation so found. Fair market value under these circumstances would of course be defined to be the value as established between a seller who was willing but not compelled to sell and a buyer who was willing but not compelled to buy.

In all that follows, it will be assumed that the guesses as to life expectancy and hence of proper or "adequate" depreciation reserves are accurate ones.

To come to the discussion of Mr. Alvord's conclusions, it is undoubtedly true as the author states that "there must be in this, as in all other questions relating to this difficult subject of valuation, a just and equitable relation between the public and the public utility owners, which is founded on rational analysis and common sense." In seeking out this relation much is gained in the way of clarity by confining ourselves at first to the consideration of simple examples. We are thus able to eliminate the many complicating, confusing, and often entirely irrelevant considerations which usually cloud and obscure the vital principles involved, when the question is studied in the light of complex cases such as commonly come before the Courts. Although it is no doubt true, as the author implies, that conclusions deduced from the consideration of simple examples will not always admit of indefinite extension to cover the complex cases met in actual practice, we seem at least to be on safe ground in assuming, as a point of departure, that a conclusion which will not withstand the most searching

Mr. Knox. analysis, when viewed in the light of its application to a simple practical example, cannot be the basic principle or relation which we are seeking. At any rate, let us grant the truth of this proposition for the moment, and observe the result of applying such a test to the conclusions reached by Mr. Alvord, beginning with his first conclusion, "that if a sinking fund or a reserve fund for depreciation is actually kept in the bank or in trust as part of the property, it should, if properly computed and accurately kept, receive the same rate of fair return as the remainder of the property 'used and useful for the public'."

If by a "properly computed and accurately kept" sinking fund, Mr. Alvord means an "adequate" depreciation reserve, as defined in a preceding paragraph of this discussion, the logic of this conclusion appears to be unassailable.

Consider, for example, a water-works plant, consisting solely of a pipe line 10 miles long. The owner of this property, we will say, buys water at wholesale at one end of the pipe line, and sells it at wholesale at the other end. The writer has in mind a private water company the property of which is substantially as described. Assume that the reproduction-cost-new of the pipe is $1 000 000, its useful life 50 years, and its scrap value nothing. No part of the line will require replacement during its 50-year life, and the pipe may be maintained in good serviceable condition throughout the entire period by making such ordinary repairs as may be properly charged to operating expense. Neglecting the fact that the creation of a sinking fund must lag one year behind the commencement of operation, the proper annual reserve to cover depreciation, estimated on a 4% sinking-fund basis, is $6 550, which amount the owner places in bank each year at 4% compound interest.

In the construction of this pipe line, the owner converted $1 000 000 of money into physical plant. As the pipe ages, it depreciates in value, and if the guess as to its useful life (50 years) proves correct, the amount of this depreciation is, as nearly as may be, represented at any time by the accumulation in the sinking fund or reserve fund for depreciation. The process may be regarded as the gradual re-conversion of plant value into cash, until at the end of the pipe's useful life the owner has all cash and no pipe line, at which time the money accumulated is again converted into physical plant through the taking over of a new pipe line which the owner has caused to be constructed. At any intermediate period of the life of the pipe, the owner's capital is represented partly by the depreciated value of the line, and partly by the depreciation reserve. The sum of the two may be considered as representing at any time the reproduction-cost-new. A purchaser can at any time afford to pay for the pipe line and depreciation fund together, a sum equal to the pipe's repro-

duction-cost-new, for by continuing the depreciation reserve in bank, and adding the proper annual payments, as these are earned, he will have available at the termination of the useful life of the property a sum equal to his original investment and to the pipe's reproduction-cost-new. This is equivalent to saying that a purchaser can at any time afford to pay for the pipe line alone a sum equal to its reproduction-cost-new less the accumulations in the sinking fund, or its depreciated reproduction cost. Mr. Knox.

Now, as all interest accretions, if the annual depreciation reserve is a proper or "adequate" reserve computed on the sinking-fund basis, must be credited to the fund, the owner enjoys no income therefrom, and, therefore, should be allowed to earn a fair return on the depreciated physical value plus the depreciation reserve, or on the reproduction-cost-new. To assert the contrary is to assert that the owner must, in spite of everything he can do, suffer a financial loss through impairment of the earning power of the capital which he has invested.

Let us now examine Mr. Alvord's second conclusion:

"That if these funds [adequate depreciation reserves] are for any reason detached or withdrawn from the property and used by the owner elsewhere, or in private gain, or even as new capital invested in the plant itself, he cannot hope, as a matter of proper protection to the public, to receive return on a reserve fund which is not actually in hand and at the same time use such funds for other personal gain."

Suppose that the owner of the pipe line operates his plant in exactly the same way as described, except that instead of actually placing $6 550 in bank or in trust each year as part of the property, he invests this amount in other ways, or even places it in a safe deposit vault or squanders it. Mr. Alvord concludes that under these circumstances rates may be equitably adjusted to yield a fair return only on the depreciated physical value of the pipe instead of on its reproduction-cost-new. This means that unless the annual depreciation reserves are correspondingly increased above the "adequate" amount previously estimated, a sliding scale of rates must be introduced, such that the amount available for interest in the tenth year of operation, after deducting operating expenses and depreciation reserve, will be approximately 92%, in the fortieth year approximately 38%, etc., of its amount in the first year of operation. As the service is now by hypothesis exactly the same as in the case where the depreciation reserve was actually deposited in bank as part of the property, the water consumer will evidently gain in reduced rates by the owner's failure to establish an actual depreciation reserve. It is equally evident that the consumer cannot gain unless at the same time the owner loses an equal amount. Let us pursue the illustration further.

Suppose that the owner, in the tenth year of operation, becomes alarmed at the dwindling profits on his water transportation business,

Mr. Knox. and begins to busy himself in devising a plan for restoring rates to the point where they will yield the same return on his capital as in the first year. He estimates that if he had from the outset maintained a depreciation fund in the bank at 4%, the accumulations in this fund would at the end of the tenth year have amounted to $78 760.56. He, therefore, sells at par from his private means seventy-nine $1 000 municipal bonds drawing 4% interest, or he sells, we will say, an apartment house valued at $78 760.56, which has been yielding him a net return of 4% per annum. He deposits the proceeds, $78 760.56, in the bank at the beginning of the eleventh year and christens it a depreciation fund. He has not lost a dollar by this transaction. The depreciation fund is his property in the same way that the house was his property. The interest yield is the same. He has simply performed the familiar operation of taking a sum of money out of one pocket and placing it in another, and yet, as a result of this simple shifting of assets, he has placed himself in a position where he may equitably charge his customer, the water consumer, we will say, an additional $10 per 1 000 000 gal. for exactly the same quantity and quality of water, delivered under exactly the same pressure as before the depreciation fund was created. Could anything be more absurd?

Mr. Alvord speaks of the use of the depreciation reserves, by the owner, "for other personal gain". It is, of course, apparent that when these reserves are proper or "adequate" reserves computed on the sinking-fund basis, it is entirely beyond the power of the owner to use them so as to yield "a private gain", unless he invests them in a way to yield a return greater than that which they would have earned if placed in the bank or in trust at the rate of interest on which the sinking or depreciation fund payments were computed.

If correctly computed on the sinking-fund basis, the sum of the annual reserves will, by hypothesis, produce an amount sufficient to provide for future replacements, only if credited with all interest accretions resulting from their conservative investment. The source of these interest accretions is immaterial. The funds are equally unproductive, as far as concerns their power to yield a "private gain" to the owner, whether they be placed in bank at 4% interest, invested in high-grade municipal bonds at 4% interest, or invested in other ways. If the owner is fortunate enough to invest them in a way to yield a return greater than they would have earned if placed in a bank or in trust, his "private gain" is represented, not by the total yield of the funds so invested, but only by the excess of this yield over and above what they would have earned if placed in the bank or in trust. To obtain this excess yield or "private gain", the owner performs services and assumes risks for which he is entitled to compensation represented by the excess yield. He has, in fact, entered the banking business and is as much entitled to a fair profit on this banking business

as he is on his water-works business. In his capacity of water-works manager, he deposits with himself, in his capacity of banker, the depreciation reserves on which he pays 4% compound interest. When the time arrives, he must return these sums with accumulated interest, the same as any other banker would be obliged to do. In the meantime he is entitled to invest them conservatively in such a way that they will yield enough more than 4% to compensate him for performing the services and assuming the responsibilities and risks which attend the banking business. Mr. Knox.

There are bankers and trust companies, many of them, that actually own and operate water companies. Is it to be presumed that these banking institutions, to avoid being "fined," must invest their depreciation reserves with some competitor, perhaps less responsible and competent than themselves? Again, the proposition is absurd.

Theoretically, the consumer has no interest whatever in the depreciation reserves or in the disposition which is made of them. He is interested only in the quality of the service. Service is what he pays for. He pays not for the quality of service which an element of plant is capable of rendering when new, or at its absolute maximum of efficiency, but for the average service which it is capable of rendering during the period known as its "useful life," when maintained throughout this life in the highest state of efficiency which is physically possible, and replaced at its termination with a new element. In the absence of certain knowledge to the contrary, it must be presumed that the owners will so maintain and so replace the plant, regardless of whether or not an actual, tangible depreciation fund is maintained in a bank—a competitor's bank. Even when depreciation reserves have been distributed to stockholders, or lost or tied up in unfortunate investments, it must be presumed, in the absence of certain knowledge to the contrary, that the owners will provide funds as required by dividend reductions, assessments against the stock, or in extreme cases by reductions in interest.

As a practical matter, it is granted that in certain cases the owner's failure to invest depreciation reserves conservatively may react on the consumer, and that to this extent he is interested in the disposition which is made of them. It is, for example, a matter of common observation and knowledge that when an owner first becomes involved in financial difficulties there exists the temptation and tendency to maintain dividends and interest at the expense of maintenance and replacements. The prevention of this is one of the functions of the regulating power.

As long as the quality of the service is actually maintained, there can exist no logical reason why the value on which the owner of a property is entitled to earn a fair return should be variable, depending on whether or not a reserve for depreciation is maintained in the

Mr. Knox. bank or in trust as part of the property. To penalize or "fine" an owner for failure to maintain an actual fund by adjusting rates to yield a fair return only on depreciated reproduction-cost, is to presume in advance of the fact that he will not, when required, and at his own cost, make the replacements for which depreciation reserves were provided. It is in effect to punish him in advance for an offense that it is suspected he may commit at some time in the future.

The points just made have been clearly stated by the Wisconsin Railroad Commission *re* City of Whitewater and Whitewater Electric Light Company, decided December 16th, 1910.* The Wisconsin Railroad Commission says:

"As it is a general rule that the reasonable return which a utility is allowed to earn covers the interest and depreciation on the actual investment in the plant, it becomes important to know what the investment in the plant actually is—that is, what is the value of the plant new. The fact that the property of the utility has diminished in value with use, as the inevitable result of depreciation, does not lessen the amount of the investment in the plant. To be sure, it may happen in the case of a given utility that money which should have gone to the establishment of a depreciation fund has been diverted to the stockholders, thereby apparently lessening their investment. If an amount equal to the difference between the value of the plant new, and the value in present condition is thus paid over to stockholders, it would appear, at first sight, that the value of the plant in present condition would be the basis on which interest returns should be allowed. But it must not be forgotten that at the expiration of the life of the plant if the money which should have been used to provide for depreciation has been paid to stockholders in the form of dividends or otherwise, the value of the plant will be nothing. Then, instead of the utility having a depreciation fund on which to draw to replace the plant, the owners will find it necessary to pay the cost of replacement, presumably from the money which they have received from the plant, but which should have been used to provide a depreciation fund. The investment in the plant then must, in general, be taken at the cost of the plant new, since although the investment may apparently be diminished by failure to provide for depreciation, and by the payment of this money to owners or stockholders, in reality the investment is not diminished, because of the necessity of replacing the plant, in the absence of a depreciation fund, from the property of owners or stockholders. Therefore, it appears that the question of valuation, which is of most importance in the case, is that of cost of the plant new, or the actual value of the total investment in the plant."

It is not necessary to state that a reduction, below reproduction-cost-new, of the value on which an owner is permitted to earn a fair return, may be logically justified only on the assumption that a virtual refund of a portion of the capital originally invested has taken place prior to the date of the valuation for rate-making purposes. It has

* Robert H. Whitten, "Valuation of Public Utility Properties," p. 366.

been shown that no such refund is effected where an owner, in lieu of maintaining an actual tangible depreciation fund in the bank, invests these reserves in other ways. There are, however, ways in which such a virtual refund may occur, and it may prove interesting to examine one or two of them. Mr. Knox.

Suppose, for example, that the owner of the pipe line, previously referred to, instead of setting aside each year in addition to operating expenses and interest an "adequate" depreciation reserve of \$6 550, sets aside a depreciation reserve, improperly computed on the straight-line basis, of \$20 000 per year. Under these circumstances interest accretions are no longer, of necessity, credited to the fund, for the sum of the annual payments themselves, will, without interest, accumulate an amount sufficient to replace the pipe line at the end of its useful life. Whether placed in the bank as an actual tangible depreciation fund, or invested in other ways, the owner may each year draw and add to his gross earnings the income from this fund without in any way defeating the object for which it was created. However it be invested, the entire fund may now be used productively by the owner "in private gain." A portion of his original investment equal in amount to the sum of the annual depreciation reserves has in effect been refunded to the owner by the consumer. It has been refunded free and clear, together with its full earning power, and if the owner is to be prevented from receiving, at the expense of the consumer, a double interest return on the same principal, rates must be adjusted to yield a fair return not on reproduction-cost-new and not on depreciated reproduction cost, but on reproduction-cost-new less the sum of the "excessive" annual depreciation reserves. This value will be less than the depreciated reproduction cost. At certain periods during the life of the pipe, it will be materially less. At the end of the twenty-fifth year, for example, the "accrued depreciation," or the sum amortized by an "adequate" depreciation reserve of \$6 550 per annum, will be only \$272 803, whereas the sum of the annual depreciation reserves, without accrued interest, will be \$500 000. The depreciated reproduction cost on that date will be \$1 000 000 — \$272 803 = \$727 197, whereas the value on which the owner should be allowed to earn a fair return will be only \$1 000 000 — \$500 000 = \$500 000, or about two-thirds of the depreciated reproduction cost.

If an owner has, during the past, set aside out of earnings an "excessive" annual depreciation reserve, or if he has, what amounts to the same thing, enjoyed excessive profits, it may appear from the preceding discussion that the ends of justice will be served by adjusting rates to yield a fair return on a value less than reproduction-cost-new. The justice of the procedure, when adopted as a retroactive or punitive measure, is, however, only apparent. The ownership of most public utility properties is constantly changing through transfers of stock,

Mr. Knox. and reprisal would in many cases fall not on the original owners, who alone could be supposed to merit it, but on the present owners who purchased in good faith, at a fair price, and reaped no benefit from past extortions.

Suppose, for example, that at the end of the twenty-fifth year, and just prior to the date of a valuation for rate-making purposes, the pipe line, previously referred to, had been sold by the original owner, A, to the present owner, B. Suppose that A, throughout the period during which he operated the line, had extorted from the public, in addition to a fair return on his capital, improper depreciation reserves estimated on the straight-line basis, which amounts he did not transfer to B with the depreciated property. A has now placed himself beyond the reach of reprisal. He has gone out of the waterworks business, taking his unholy profits with him. He has sold his plant for its market value which, for the purposes of this discussion, may be assumed to be fairly represented by its depreciated reproduction cost at the date of sale ($727 197). He could not have sold it for more however moderate his early charges for depreciation had been. B has purchased the property in good faith and for a fair price. It might now be argued superficially that as far as B is concerned, the price paid represents his total investment, and that the adjustment of rates to yield a fair return on this purchase price will impose no hardship on him; but this is not the case. The fallacy in the reasoning, though perhaps not obvious, lies in the assumption that the money which B has paid to A for the physical property represents his total investment in plant. It does not. A has sold to B a depreciated physical property plus a liability. The liability consists in the obligation which B incurs of replacing the depreciated pipe line at the end of its useful life. To place himself in a position to meet this obligation when it matures, B must, on the date of the purchase, set aside in bank at 4% interest an amount ($272 803) equal to the accrued depreciation on the plant. If he fails to do this, he, nevertheless, automatically, by the assumption of the obligation, ties up an equal amount of his outside investments as far as concerns their power to yield him a "private gain." In any case his total investment is not represented by the sum of money paid to A, but by this amount plus the accrued depreciation. Whatever disposition he makes of the depreciation reserve, if it is only an "adequate" reserve as previously defined, B will inevitably suffer a financial loss unless allowed to earn a fair return on $1 000 000, the reproduction-cost-new. Any reduction in this value will inevitably work a great injustice by causing B to suffer for the sins of his predecessor.

B, it is true, will be protected against loss, if at the same time that rates are adjusted to yield a fair return on depreciated reproduction cost, he is allowed to set aside each year from earnings, an "exces-

sive" depreciation reserve sufficient to reproduce the depreciated plant value ($727 197) at the end of its 25 years of remaining life. It may indeed be laid down as a general rule that where past depreciation reserves with interest accretions have been adequate to take care of the accrued depreciation, and future reserves are made sufficient in themselves to amortize sums which will provide against all subsequent deterioration, rates may fairly be predicated on the depreciated reproduction cost at the date of the valuation for rate-making purposes.

Although there is one notable exception which the writer will discuss later, the adoption of this rule will usually result in difficulties and confusion in accounting, with no attendant benefits either to the consumer or to the owner. Decreasing the amount allowed for interest and at the same time allowing a corresponding increase in amounts allowed to offset depreciation is a mere juggling of figures. An annuity sufficient to amortize the depreciated value of a property at the termination of its remaining life expectancy will become an "excessive" annuity on the day the first replacement is made, and generally the depreciated reproduction cost will be modified as a result of each replacement.

On the other hand, if we neglect the fluctuations in the prices of labor and materials, reproduction-cost-new will remain an unvarying quantity as long as no additions are made to the property, and the "adequate" annual depreciation reserve, estimated in the customary way, will remain at all times a constant percentage of the unvarying reproduction-cost-new. The reproduction-cost-new having once been ascertained may subsequently be kept up to date merely by adding the cost of extensions as these are made, and the "adequate" annual depreciation reserve, having once been ascertained, may be kept up to date, as extensions are made, by adding sums sufficient to reproduce the cost of each such extension at the termination of its useful life.

If past depreciation reserves, with interest accretions, have been no more than adequate to offset the accrued depreciation on the date of the valuation, either method will result in justice both to the consumer and to the owner. The choice of method is merely a question of convenience, and generally it will be found most convenient to predicate rates on reproduction-cost-new, at the same time limiting the annual depreciation reserve to the "adequate" annuity, as previously defined.

The exception to which the writer has referred is presented by the case, briefly touched on by Mr. Alvord, and also discussed by Henry Floy, M. Am. Soc. C. E.,* in which certain large properties, after suffering an initial depreciation, reach a stable stage in which they may afterward be maintained by the expenditure of certain sums each

* "Valuation of Public Utility Properties," p. 194.

Mr. Knox. year for replacements, the cost of these replacements being charged to operating expenses the same as ordinary wear and tear. To quote Mr. Floy:

"The Receiver of the Third Avenue Railroad in New York City, operating a large property having numerous physical elements so that all deterioration became simply 'wear and tear' and a part of operating expenses, declined to obey the order of the Public Service Commission and provided no depreciation fund whatever, simply removing deterioration when it occurred and charging it as maintenance in operating expenses."

If past depreciation reserves, with interest accretions, have been no more than "adequate" to offset "accrued depreciation," all subsequent depreciation on a property which has reached this stage of stability, may properly be taken care of in either of two ways:

1st.—By continuing the "adequate" annual depreciation reserve; in which case rates must be predicated on reproduction-cost-new.

2d.—By providing annually, in the manner adopted by the Receiver of the Third Avenue Railroad, a sum sufficient in itself to take care of all subsequent depreciation, in which case rates should be predicated on depreciated reproduction cost.

By the second method the annual depreciation reserve is in excess of an "adequate" reserve, as previously defined, and it is for this reason, and for this reason alone, that rates may be equitably adjusted to yield a fair return on a value less than reproduction-cost-new.

Consider, for example, a plant composed of five elements of equal value and variable life expectancy as shown in Table 4.

TABLE 4.

Element. (1)	Reproduction-cost-new. (2)	Life expectancy, in years. (3)	Annual depreciation reserve to refund reproduction-cost-new at termination of useful life, sinking-fund basis. (4)
A	$1 000	25	$24.01
B	1 000	30	17.83
C	1 000	35	13.58
D	1 000	40	10.52
E	1 000	45	8.26
Total, or average...	$5 000	....	$74.20

Assuming the scrap value to be nothing, the orthodox way of providing for the future depreciation on such a plant would be to set aside each year from the beginning of operation sums sufficient to amortize the reproduction-cost-new of each element at the end of its

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useful life. The annuities so required, estimated on a 4% sinking-fund basis, are given in Column 4 of Table 4. The total annuity required is \$74.20. At the end of the twenty-fifth year, element A must be replaced, and the cost will be met by withdrawing from the depreciation fund the \$1 000 accumulated by the annuity of \$24.01 provided for this purpose; but, as the new element, A, will also have a life expectancy of twenty-five years, at the end of which it must again be replaced, the annuity of \$24.01 provided to cover depreciation on element A must be continued and the total annuity will remain \$74.20 as before. It will remain so indefinitely as long as no additions are made to the plant. Whether it be placed in bank or in trust at 4% interest, or invested in other ways, the annuity of \$74.20 will, together with all the interest accretions with which it may fairly be credited, just take care of future depreciation, and no more.

Assuming that the owner actually makes such an annual reserve, the amount amortized at the end of the twentieth year will be \$2 209.65, which amount may be taken to represent the accrued depreciation on that date. The depreciated reproduction cost at the end of the twentieth year will then be \$5 000.00 — \$2 209.65 = \$2 790.35. As the owner can by hypothesis derive no "private gain" from the depreciation fund, however it may be invested, he must, so long as he provides for depreciation in this way, be allowed to earn a fair return on reproduction-cost-new.

Now assume that at the end of the twentieth year the owner perceives that 5 years hence he must replace element A at a cost of \$1 000. He realizes further that at the end of each succeeding 5-year interval he must replace one element of his plant at a cost of \$1 000. He estimates that the annuity necessary at 4% compound interest to produce \$1 000 at the end of 5 years is \$184.63, and he elects to take care of all subsequent depreciation by reserving this amount each year from earnings. The future cost to the consumer under this scheme will be increased in the annual sum of \$184.63 — \$74.20 = \$110.43, and he must, obviously, if he is to avoid loss, obtain a reduction in the plant value or capitalization on which the owner is allowed to earn a fair return. Nor will the owner under these circumstances suffer any loss by such reduction in value. He has, in a sinking fund or otherwise invested, at the end of the twentieth year \$2 209.65, equal to the accrued depreciation on his property as of that date. Under the first or orthodox method of providing for replacements, this sum, together with its earning power, was necessary to the proper provision for depreciation. Under the second scheme, neither the sum nor its earning power is so required. On the date on which the owner changes over from the first to the second method of providing for future depreciation, this fund is released or set free, and may henceforth be used productively by the owner in any way he sees fit. The consumer has in effect, by permit-

Mr. Knox. ting the change in method, refunded to the owner, free and clear, a portion of his original capital equal in amount to the accrued depreciation on the property. Henceforth the owner's investment in plant is equal, not to the reproduction-cost-new, but to the depreciated reproduction cost, and this therefore is the sum on which he should be allowed to earn.

The case presented is only a special one under a general rule in which the interests of convenience are served by lowering below reproduction-cost-new the value on which the owner is allowed to earn a fair return, at the same time increasing above the "adequate" depreciation allowance, the sums annually set aside out of earnings to offset deterioration. It is in no sense an exception to the general law that where depreciation reserves are only "adequate" reserves, as previously defined, rates must be predicated on reproduction-cost-new.

The application of the general law will usually be more convenient, and will always result in justice both to the public and to the owners, unless we regard as an exception the case in which the owner has actually, during the past, set aside from earnings as an offset to deterioration an annual sum more than sufficient to take care of the depreciation which has occurred, or, what amounts to the same thing, unless the owner has actually during the past enjoyed excessive profits for which he is now to be penalized.

A reduction in the value on which he is now permitted to earn may be logically justified on this ground only in the event that the present owner has benefited from these extortions. Even on this supposition, the logical and scientific procedure is to take as our point of departure the basic principle that in the absence of such complicating considerations, the owner should be allowed to earn on reproduction-cost-new; then to apply proper and specific allowances for such irregularities as the history of the plant may disclose. In this way "the punishment" may be made "to fit the crime," and we will always know, at least by how much and why, we have lowered or increased below or above reproduction-cost-new the value on which the owner is entitled to earn a fair return, for it must not be forgotten that it is a poor rule which does not work both ways, and if an owner is to be penalized for "excessive" depreciation reserves or exorbitant profits made in past years, consistency demands that he be compensated for his past failure to provide "adequate" depreciation reserves and fair profits.

If, for example, the owner in the case just discussed, has during the first 20 years, in which the necessity for replacements was not conspicuous, made no reservations whatever for depreciation, but has been satisfied to earn a fair return on his investment in addition to operating expenses, in the expectation that future replacements may be provided for as a part of operating expenses, when the occasion arises it is evident that his capital will be impaired by an amount ($2 209.65) equal

to the accrued depreciation if rates are adjusted in the twentieth year to yield a fair return only on depreciated reproduction cost. In this instance, however, the loss is largely the result of his own folly, in failing to make provision at the proper time for the deterioration which was taking place. Mr. Knox.

He would be obliged to incur the same loss if he undertook to sell the property in the twentieth year, as it cannot be presumed that the physical elements of the plant could be sold on that date for an amount greater than their depreciated reproduction cost.

Mr. Humphreys, in discussing this question in the paper to which the writer has previously referred, quotes an amusing illustration taken from what he characterizes as the most logical paper on this subject he has ever read. The quotation is credited to a brief by Charles F. Mathewson, of the New York Bar, in the case of Kings County Lighting Company *vs.* The Public Service Commission for the First District of New York. Mr. Mathewson was the trial lawyer for the Company in the New York Consolidated Gas Company's case. The quotation follows:

"The proposition [to deduct 'accrued depreciation' in valuing plants in rate-making] is so absurd on its face that it hardly needs discussion to show its fallacy. Why, aside from the question of 'confiscation,' should consumers, for exactly the same service, equally efficiently rendered, expect to pay less in the sixth year than in the first year, merely because some items of plant will (viewed at the sixth year) require replacement at a date in the future then nearer than such date was at the beginning of operation? As well might it be claimed, to repeat a homely illustration, that a farmer should regulate the price of the eggs which he sells, by the age of the hen which lays them—reducing the price of the product as the hen gets on in years. The reason he does not is that the service efficiency and operating value of the hen, as evidenced by the quality of the eggs which she lays, are not impaired by the fact that her life is advancing. That advancement may concern the farmer and possibly concerns the hen; but it in no manner affects the value of the eggs to the consumer, or justifies him in demanding them at a lower price than he paid at an earlier period of her life. The consumer of the eggs must expect to pay a sufficient price to afford a return to the farmer on his total investment in the hen during her life, *plus* enough more to enable the farmer on her death to replace her, and thus keep his investment unimpaired. A farmer could hardly be expected to invest in hens for the purpose of supplying the public with eggs, if for a portion of their life he was to receive a return on only a third or a half of his investment; and any such rule would simply compel the public to go without eggs until the regulating power (if such there were) saw fit to revise its reasoning. There is absolutely no difference in the economic principles applicable to the operation of a gas plant and the operation of a hennery, so far as concerns right to return on capital; and what is absurd in one case is equally absurd in the other. The fact that the rate of return in the one case is subject

Mr. Knox. to reasonable regulation, and not in the other case, has no bearing on the main proposition."

A consideration of this illustration in the light of the principles thus far set forth indicates that it is not of universal application except as an illustration of the fundamental principle which always holds where complicating considerations are absent. It presumes that the farmer charges no more than enough to afford a return on his total investment plus enough to keep his investment unimpaired. If, as some consumers think, the farmer charges enough in addition to a fair return on capital invested, to replace the hen several times over, prior to her demise, the absurdity of expecting a reduction in the price of eggs, even in this case, from a purely theoretical point of view, is not quite so apparent.

It is readily admitted that the preceding discussion has been carried on largely from a theoretical point of view; but, considering the present confusion of ideas on the subject, this theoretical treatment of the question appears to be fully justified. Sound practice, in this as in other engineering matters, must rest on a firm foundation of sound theory, and the establishment of this theory has something more than academic interest.

In concluding it may be well to point out that too much importance may easily be attached to the fact that the Supreme Court of the United States has in certain instances held that rates should be adjusted to yield a fair return only on depreciated reproduction cost. It is of course true that the last word in these matters rests with the Courts, and that Courts are much given to following precedent; but it must not be forgotten that Court decisions are usually rendered with a view to dispensing justice in the particular case before the Court. It has been shown that there may be circumstances in which the ends of justice are served by allowing an owner to earn a fair return on depreciated reproduction cost. Whether or not the cases so decided by the United States Supreme Court fall in this category we do not presume to say, but even this is unimportant because it cannot be presumed that the Courts will seek to perpetuate what can be demonstrated to be a wrong merely to avoid going against their own precedents. The question cannot be regarded as finally settled until it is settled equitably, and when, as in the present case, such a wide diversity of opinion prevails among those who have spoken most authoritatively on the subject, the attempt to reason the problem out *a priori* seems to be fully justified.

Mr. Gandolfo. J. H. Gandolfo, Assoc. M. Am. Soc. C. E. (by letter).—In a discussion dealing with appraisal, valuation, depreciation, rate-making, or kindred subjects, the argument is frequently advanced that no consideration must be given in cases in which all matters pertaining thereto have not been conducted in a fair, honest, proper, and business-

like manner. If the human race was still living in the Garden of Eden; if Pandora had never allowed all those troubles to escape into the world; if there were no such things as selfishness, greed, hate, envy, revenge, and a host of other evil passions; in short, if the great struggle for existence throughout the world (and this struggle applies to all plant and animal life, as well as to all members of the human race) did not exist, then it would be possible to discuss such questions purely academically, and arrive at such a course of conduct for all parties concerned as would lead the human race along a broad and happy highway of peace, prosperity, and contentment. Under these circumstances, the very necessity for such things as public service commissions, appraisals, hearings, judicial reviews, and hosts of kindred matters would not exist. Mr. Gandolfo.

Unfortunately, however, the millennium is still as far away as ever. It is just such elements as those described in the first part of the foregoing paragraph that go to make up the human mind, and therefore such elements must be taken into consideration and given careful attention in such discussion. In other words, it is the much used term "human nature" that must be studied and reckoned with, and taken into account, in such investigations. Facts on record relating to bond and stock issues, and to general methods of financing and of business, especially in the railroad world, have made history, showing most conclusively that human cupidity and the still baser human passions have played a most important rôle in these matters from the year 1838 to the present day, resulting in the loss of millions of dollars to honest investors, both large and small.

Mr. Alvord says, on page 2048:* "In other words, no amount of accounting or theorizing can make good a depreciation fund which is actually not in hand". In the succeeding pages, however, he seems to lean to the idea that a sinking fund provided to take care of depreciation or obsolescence can be invested in some other enterprise and still remain a sinking fund, and goes into a great deal of detail to explain why this is so. For example, on page 2056,* he says: "* * * for instance, such moneys may be invested as new capital in new construction and extensions in the very plant itself to great advantage * * *."

Webster defines "sinking fund" as follows: "(*Finance*), a fund created for sinking or paying a public debt, or purchasing the stock for the government". The Century Dictionary and Cyclopædia gives the following: "A fund formed by a government or corporation for the gradual 'sinking', wiping out, or reduction of its debt, by various devices for the accumulation of money." Funk and Wagnalls' Standard Dictionary says: "A fund instituted and in-

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Mr. Gandolfo. vested in such wise that its gradual accumulations will enable it to meet and wipe out a debt at maturity".

In view, therefore, of the very fundamental idea and *raison d'être* of a sinking fund to cover depreciation and obsolescence, it is an absolute impossibility to consider it as being invested in any other enterprise, except as a bank or trust company would invest such funds. The moment such a sinking fund is used by the owner of a plant or a board of direction for any purpose other than that for which it was originally set aside, the fund, as a sinking fund, ceases absolutely to exist. No longer can it be counted on as needed to replace the plant. It is gone, absolutely and entirely, as far as its original purpose was intended, and no amount of bookkeeping or argument can make anything else out of this fact.

On page 2054* Mr. Alvord says: "* * * there would be no difficulty or hazard in relying on the ability of the owner or owners to replace the sinking fund from outside sources * * *." On page 2055* he also says, "It may be objected that it is a hardship on owners of public utilities, who are able and willing to replace in the property such portion of the renewal fund at any moment * * *", and on page 2056,* "* * * the owner must be willing and ready to finance the replacements and depreciation fund, whenever the occasion therefor arises * * *." Admitting for the moment the willingness, suppose, when the time comes, that the owner is not able to do so? What then? And further, suppose that such a fund has been invested in some other way, or has been appropriated for some other purpose, and then the parties at fault, although fully able to do so, are not willing to make restitution when the fund is needed for its original purpose. Who is to make them do so?

In connection with this matter of the investment of a sinking fund, it may be argued that the bank or trust company in which such a fund would be deposited, to draw interest at ordinary rates, would invest these moneys in some way, and therefore there is practically no difference in the end, than if the owners invested the fund direct. There is a very great difference, however, which may be summed up as follows:

(1) The owner is an interested party, and as such, is not capable of investing the fund to as good advantage as the bank. He would be more apt to have bias or prejudice, good or bad, and it would be absolutely impossible for him to look at things in the free, untrammeled way that the officers and board of direction of a bank are supposed to do.

(2) The fund in question is not the only resource possessed by the bank. There are dozens of others, including capital and surplus, and

* *Proceedings*, Am. Soc. C. E., for November, 1913.

if one fails or is in such form that it cannot be realized on at once, there are others that can be. In this way, the bank may be likened to an equalizing pond or reservoir for the benefit of all its depositors. The drain in one direction may be heavy, while that in others is light, and at the same time there is a constant supply from all sources. This condition could not possibly exist if the fund was directly invested by the owner. Mr. Gandolfo.

(3) The bank or trust company is in its turn a "Public Service Utility", subject to the most careful scrutiny by the State, and thus an additional safeguard is thrown about all sums placed here in trust. If the fund was privately invested by the owner, there would be no such safeguards.

Further, if the owner, as stated by the author, must always be ready to make good the depreciation and obsolescence, he in his turn must have funds immediately available with which to do so. As, following out this line of argument, he would ordinarily have all his personal funds invested, it would then be necessary for him to maintain some fund for this particular purpose. Wherein lies the difference, then, under such conditions, whether such a fund is held by the original party, or must go through several hands, except the far greater risk and uncertainty in regard to its ever materializing at its original source when needed.

As far as investing a sinking fund in extensions to the plant, the moment it is so invested, it becomes "Capital", subject at once in its turn to depreciation, and is no longer "Sinking Fund". Further, if the same rate of profit is allowed on the sinking fund as on the plant (and, under conditions as hereinafter described, there seems to be no question but that there should be), then there is no difference as to whether the fund is invested in new construction or not, as far as the returns to the owner are concerned.

As for a concrete example illustrating much of the foregoing, there is one in the New York, New Haven and Hartford Railroad. The funds of this road, which should have been used to take care of depreciation, obsolescence, and improvement, within the last few years have been used for other purposes, the details of which it is needless to go into here. When, a short time ago, on account of a series of accidents that cost the lives of scores of people, an investigation by the State disclosed the fact, among others, that sections of this road were still equipped with signals of a type which had been discarded as obsolete by other roads some fifteen or more years ago, and it therefore became imperative to use for improvements the funds that had been otherwise "invested", did the parties who had appropriated them make any move to replace them? When public opinion, however, at last aroused, demanded that something be done, $67 552 000 of 6% convertible debenture bonds were offered to the general public, in order to take care of some $46 023 750 of obligations which are to mature within com-

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paratively short periods, the remainder being to pay for financing and to obtain funds to make good the depreciation and obsolescence. The issue of these bonds has been opposed by the present bondholders, and the offering of them is now awaiting the decision of the Court. In place of this, approximately $45 000 000 of short-term notes were issued, to pay for the immediately maturing obligations, and also to provide for pressing needs due to depreciation and obsolescence.

Further examples, such as the foregoing, might be given in detail, but there is very little use or profit to be gained by anybody in arguing theoretically about a question of this kind, when undeniable facts and occurrences are on record before the financial world and the general public.

As the author points out, at no time should a depreciation and obsolescence fund amount to a very large percentage of the cost of the plant, on account of the constant withdrawals from it to maintain the plant in first-class and up-to-date condition. If such a fund is actually in hand, that is, actually in trust, there is no question but that rates should be allowed on it, just the same as on any other item of the plant. If the plant is being kept in repair and up to date by constant withdrawals from the fund, there seems to be no reason that rates should not be allowed on the full invested capital of the plant. This matter can be treated in two ways, which lead to the same result: The property can be appraised at its full cost, without depreciation, and returns can be allowed on this figure, no special mention being made of the fund; or the property may be appraised at its cost, less depreciation, and returns may be allowed on this, and also on the depreciation fund. Of course, it is assumed that the sinking fund is always approximately equal to the actual depreciation at any time. A sinking fund, provided for depreciation and obsolescence, and actually in hand, is part of the plant, and should be treated as such.

The necessity of keeping a public utility property in first-class condition by constant withdrawals from the sinking fund can be further illustrated by the following theoretical case. Suppose that the life of the physical property has been determined, and then a depreciation fund is created which, at the end of the period, will just return 100% on the investment. At the end of this period, the plant, like the deacon's famous "one-hoss shay", would theoretically vanish into thin air, leaving the sinking fund in its place. If a fund under such conditions was allowed to accumulate, then the theory that a public service utility must render adequate and constant service to the public is untenable, and cannot be upheld, as the owners would have a perfect right to allow the plant to deteriorate gradually to nothing, or to wipe out the utility at such time as they saw fit, and leave the public without its benefits.

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This question of constantly keeping up the depreciation of a property is referred to in Knoxville *v.* Knoxville Water Co., 212 U. S., 1, Jan. 4, 1909, which says that the Company "* * * is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years, the original investment remains as it was at the beginning". That public service corporations are expected to keep the plant up to date, without allowing any excessive accumulation of a sinking fund, is shown by the opinion of Judge Evans in Cumberland Telephone and Telegraph Co. *v.* City of Louisville, 187 Fed., 637, April 25, 1911, and that of Judge Haight in People *ex rel.* Manhattan Railway Company *v.* Woodbury, 203 N. Y., 231, Oct. 17, 1911.

On page 2055,* Mr. Alvord argues that if a sinking fund has been withdrawn from the plant and used for other purposes—or, as he puts it, for private gain—then the owners are not entitled to rates on it. The writer fails to see why it is necessary to argue on such a point as this, as it is a self-evident fact. If rates were to be allowed on something in the way of a sinking fund that does not exist, then it would be just as logical to remove an entire battery of boilers (or any other part of a plant), erect and use them in some other place, and then claim rates on their value, on the ground that the owners would replace them if needed. Thus the argument might be continued *ad infinitum* until the best part of a plant had been removed and rates were still being allowed on it as originally installed.

On page 2060,* under the 5th conclusion, the author says: "That though it is not now considered improper to use reserve or sinking funds allowed for depreciation for private gain * * *". The writer hardly thinks that Mr. Alvord can have realized what he was saying when he made this statement, or else that he did not mean what he says. If a private individual owns a plant himself, and sets aside a sinking fund for depreciation, and then after a time withdraws this fund and uses it for some other purpose of his own, but still calculates that he has a depreciation fund, he simply deceives himself—and there is no deception so fatal as self-deception. If, on the other hand, the concern is a stock company, and a sinking fund for depreciation and obsolescence has been created, and then the officers, directors, or managers should use this fund for their own private gain, it would amount to misrepresentation and fraud to both stock and bond holders on the one hand, and misappropriation and embezzlement of funds on the other, both of which are criminal offenses under our penal code. Furthermore, even if a board of direction did use such a fund in some other way, even if for the direct benefit of the stockholders, it is very doubtful if, under a strict code of ethics, it would have a moral right

* *Proceedings*, Am. Soc. C. E., for November, 1913.

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to do so, without first having obtained the majority vote of the stockholders to such a venture; although, in such a case, the strict letter of the law would doubtless uphold a board in such a proceeding.

In conclusion, it may be said, therefore, that a sinking fund to cover depreciation and obsolescence must be kept constantly in hand, and be available at all times; that constant withdrawals must be made from this fund to keep the plant in first-class and up-to-date condition (that is, 100% efficiency and value); that such a fund cannot be invested in other enterprises, as then it is no longer available for its purpose, and does not exist; and that, unless such a fund, or its equivalent in some form or other, exists, the plant is not entitled to full cost or replacement value, but must be appraised on a basis of cost or replacement value less depreciation.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

By Messrs. A. C. Janni and L. J. Mensch.

A. C. Janni, M. Am. Soc. C. E. (by letter).—Before entering into any discussion on the opportunity and legitimacy of the assumption on which this work hinges, the writer wishes to call the author's attention to a little mathematical "license" which he took in simplifying the expression, $P\,Q = \dfrac{v \sin.\alpha}{\cos.(\alpha + \beta)}$,† because it undermines, perhaps, the whole mathematical "make-up" of all the formulas which depend on that *"lapsus calami"*. Mr. Janni.

There is no doubt that, as α tends toward zero, $\sin.\alpha$ may be substituted for α; this simplification being entirely legitimate; but, if α tends toward zero, the expression $\cos.(\alpha + \beta)$ does not tend toward $\cos.\beta$, by any means.

The author knows that $\cos.(\alpha + \beta)$ is purely a symbol of a certain operation to be performed, and is not an actual value; its value is given by $\cos.\alpha \cos.\beta - \sin.\alpha \sin.\beta$, and it is on this latter that he may operate his simplification. Thus his $P\,Q = v\,\alpha \sec.\beta$ should become $P\,Q = \dfrac{v\,\alpha}{\cos.\beta - \alpha \sin.\beta}$, when it is assumed that α tends toward zero, to be a mathematically correct simplification.

Concerning the assumption adopted by the author in order to find a "workable formula" for the compressive stress in concrete, and

* This discussion (of the paper by William Cain, M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† See foot-note, *Proceedings*, Am. Soc. of C. E., for November, 1913, p. 2070.

Mr. Janni.

also some of his statements, the writer cannot quite agree. Two questions are before him, namely:

1. Do we need that formula?
2. Is that formula worked out correctly?

It is the writer's opinion that neither of these questions can be answered in the affirmative.

Theory is quite competent to take care of a design of a wedge-shaped beam, if necessary, despite what the author says. If the cross-section of a wedge-shaped concrete beam is rectangular and the angle, β, is not large, the common formula for bending is sufficiently good to give the maximum compressive stress in the concrete.

It is a fact that the formula entails the truth—which, by the way, is not a postulate—demonstrated by Saint-Venant, that all bending stresses at any point act perpendicularly to the cross-section; but this, in the case where β is not very large, does not prevent the formula from giving a result sufficiently near to the truth to insure the maximum stress.

That the direction of the maximum stress, in the case of Fig. 1, is parallel to the top, $N\ M$, of the beam, and on which the author lays so much stress, is, nowadays, mathematically ascertained.* In the case of a dam design, that direction is of capital importance, and the failure to recognize it, or the overlooking of it, in the design of some dams, has often been the cause of loss of thousands of innocent lives and, perhaps, millions of dollars. In a wedge-shaped beam, however (or in a counterfort), the conditions are such that the direction of the maximum compressive stress can be ignored in most cases.

If the cross-section of the wedge-shaped beam is not rectangular, then, with proper limitations, the usual formula to determine the principal stresses, reported in many textbooks, can be used. This formula is, precisely, designed for a beam by taking as unknown quantities certain inclined stresses, called "principal stresses", instead of the normal stresses in that section, and, really, this is the most logical way of designing a beam, although the common practice, for opportune reasons, follows that of the bending and shear formulas.

Winkler, in 1867, as far as the writer knows, wrote on this subject, and, after him, Culmann, Ritter, Guidi, and others treated the same question; so that the conception cannot appear novel, as the author seems to think, at least judging by some of his remarks.

If, finally, the inclination of $N\ M$, Fig. 1, is large, then Grashof's formula, with due limitations for the case of reinforced concrete, may be used.

In view of all the foregoing considerations, the writer thinks that there is no need of a formula for the case in point, and that, after all

* M. Levy, *Annales des Ponts et Chaussées*, 1897; and F. Platzmann, *Ueber den Querschnitt der Staumauern*, Leipzig, 1908.

the arbitrary assumptions of the author in reaching his "workable formula", admitting its correct mathematical "make-up", its results are much farther from the truth than those given by the existing formulas, which, in all cases, are very reliable. Mr. Janni.

It is necessary, accordingly, to take into consideration the author's assumption in finding that formula, and, as a consequence, its mathematical reliability.

As the writer said before, it is very well known that the extreme top compressive stress is parallel to the beam top as, disregarding the friction on the soil, Fig. 1, the maximum tensile stress is parallel to the bottom, *L I*.

The author states that the maximum tensile stress at that region of the beam would be parallel to *L I,* if the beam were a homogeneous one.

Even if the beam were homogeneous, if friction is not disregarded, the maximum tensile stress, at the bottom of the beam, is not parallel to the bottom face.

When the beam is heterogeneous, as in the author's case, equilibrium conditions do not change, if friction is included; the only difference being that, as usual, the tension in concrete is disregarded, so that the designer is inclined to disregard its direction, also, unless it is not a case of scientific research.

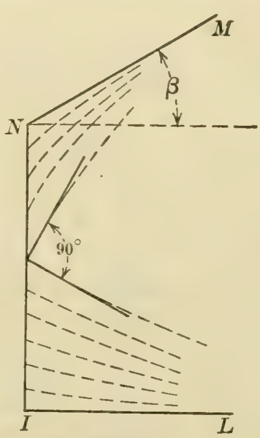

FIG. 9.

Now, the maximum compressive stress being parallel to the top, and the maximum tensile stress (always disregarding friction) being parallel to the bottom, the intermediate bending stresses, the author says, change their direction gradually from *N M* to *L I,* Fig. 1. As a matter of fact, those intermediate bending stresses do not change their direction gradually in the way the author means. Fig. 9 gives an idea of the behavior of the internal stresses in the cross-section, *N I,* on account of the angle, β. The author, however, states that, for the purpose of finding "workable formulas", he must assume that the intermediate compressive bending stresses be parallel to the maximum compressive bending stress. Such an assumption cannot be justified, when it is well known how those stresses act, and a designer knows how to determine them. Their direction, which is not constant for all, is a function of the angle, β, and not of cos. β, and their position is farther and farther from the author's assumption as β increases.

Apart from this assumption made by the author, which does not seem justifiable to the writer from any point of view, it is interesting to study the formula itself.

Mr. Janni.

Formula 8, which, according to the author, gives the maximum compressive stress in the concrete, will be taken into consideration. For the sake of clearness, the writer will compare the theoretical formula for bending in a beam of constant cross-section, which, when β is not very large, may be used without any fear of going astray, with the author's formula for the same kind of stresses in a wedge-shaped beam, using for both the same lettering, Fig. 10.

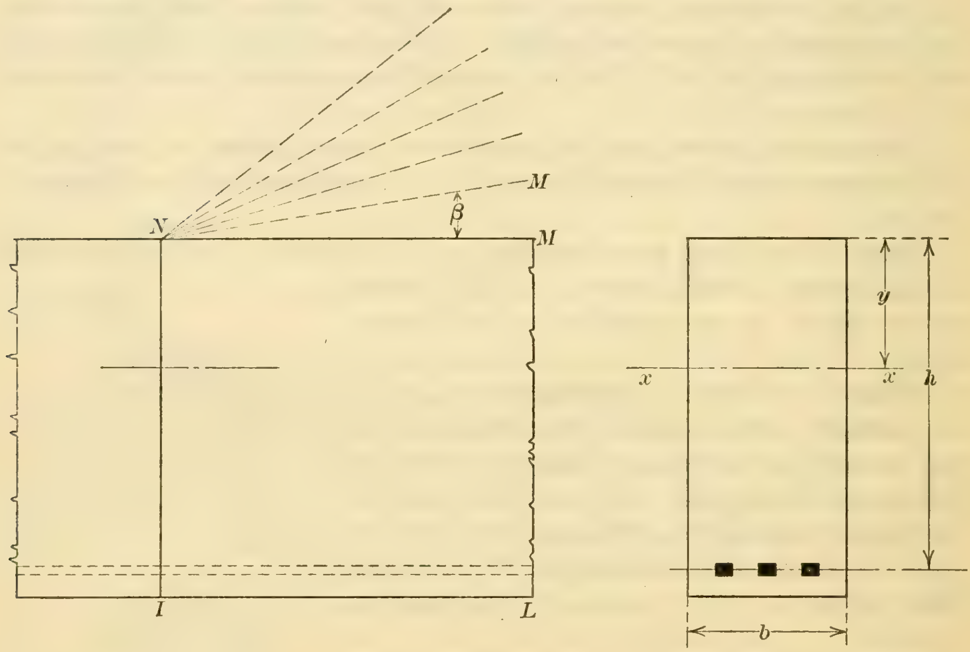

FIG. 10.

The theoretical formula for bending in a beam of constant rectangular cross-section is

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}\,y\right)} = Q \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

The author's formula for bending in a wedge-shaped beam is

$$f_c = \frac{2\,M}{b\,y\left(h - \frac{1}{3}\,y\right)\cos.^2\beta} = \frac{Q}{\cos.^2\beta} \ldots\ldots\ldots\ldots (23)$$

It is plain, as the author says, that Formula 23, for $\beta = 0°$, becomes Formula 22.

Formula 23, however, as given by the writer, presents some other information rather interestingly.

Suppose that $N\ I$, Fig. 10, be a constant section, and that the line, $N\ M$, rotates about N, the angle, β, increasing gradually while M (external moment), keeps constant, the section, $N\ I$, according to

Formula 23, becomes weaker and weaker, until, when $\beta = 90°$, it has no resisting power whatever. Mr. Janni.

The writer does not think any comment necessary, except to recall the closing words of the paper: "The writer believes that he has effected a practical solution which is on the side of safety and may commend itself to the practitioner."

It is to be added, however, that Equation 8 is not the only equation by the author which leads to such conclusions; Equation 7, for instance, and its derivative equation for "rough computations," as the author designates it, are of a similar statical consistency.

Equation 3 has been worked out to enable the finding, according to the author, of the position of the neutral axis.

It is not clear to the writer, however, why the author gives a rather long equation (which, by the way, is not correct), when there are already several plain methods of finding that axis in a reinforced concrete beam.

The writer thinks it would be necessary for the author to devote a few words in justification of his point of view concerning Equation 3.

The writer, in closing, would like to ask the author the reason for his statement that "The unit stress, v, acts parallel to $N\ N''$". The writer has the impression that this statement, as put by the author, is not another of his assumptions, but a true and proper theorem concerning the equilibrium of internal forces in a wedge-shaped beam; and, as such, he would like further explanation.

L. J. Mensch, M. Am. Soc. C. E. (by letter).—In the discussion of a paper on reinforced concrete construction before this Society several years ago, the statement was made that the counterforts of retaining walls do not act as beams. This alone would show that Professor Cain's paper is important. It also seems that the writers of textbooks have neglected to devolop theories of wedge-shaped beams, although quite a number of such books contain theories nearly identical with that of Professor Cain, for curved beams such as crane hooks, arched girders, etc. Mr. Mensch.

Professor Cain points out that an engineer is apt to overlook the fact that the compressive forces must be multiplied by $\cos.^2\beta$ when entered in the elastic equation, instead of by $\cos.\beta$ which one would be likely to do without going deeper into the matter. Probably a great many mistakes are made on this line, yet it is strange that they are made for the compression side of the beam, though on the tension side it is quite natural to multiply the actual cross-section (which is the $\cos.\beta$ value of the section made by the plane, $I\ N$) by $j\ d\ \cos.\beta$ to find the moment of the tensile forces.

For compression members, in trusses with inclined members, engineers are also accustomed to consider only the cross-section at right

Mr. Mensch.

angles to the direction of the stress; therefore, it appears to the writer that only carelessness or incompetence is the cause of the mistake complained of. Professor Cain's theory stands and falls with the fact that plane sections remain plane after deformation.

His assumption that the compressive forces are parallel to the compression face is certainly permissible in all cases where the percentage of reinforcement is low, say, less than 1%, and errs only slightly on the side of danger.

The law or assumption of plane sections remaining plane after deformation is certainly one of the greatest inventions of modern engineering, and enabled us to obtain a great number of simple working formulas, where formerly rules of thumb were used, yet the law is not universal, and is strictly proven only for straight beams of high-grade iron of rectangular section, within the elastic limit, and never for the neighborhood of ultimate load.

Professor Schule, of Zürich, about 12 years ago, proved that this assumption does not hold good for reinforced concrete beams, even under light loads, and why engineers all over the world have utterly disregarded these important tests is certainly strange.

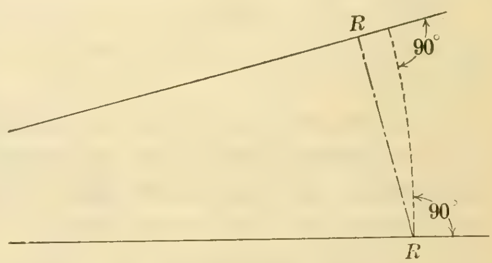

FIG. 11.

It is a well-known fact that the plane-section theory for plain concrete beams gives results which vary just 100% from actual tests, and still the same theory is applied to reinforced concrete beams. R. L. Humphrey, M. Am. Soc. C. E., in the discussion of his painstaking tests of 343 reinforced concrete beams, declares that the compressive stresses in the concrete and the tensile stresses in the reinforcement often differed 50% from the theory. In the face of such facts it may be expected that Professor Cain's table will differ just as much from facts, although it is much better than relying on a mere guess.

Tests of cast-iron beams prove that such wedge-shaped beams fail on a curved surface which is tangent to perpendicular lines to both faces of the beams, as shown in Fig. 11. For this reason the writer has advised* calculating such a counterfort for a section, *R R*, like a common beam, which greatly simplifies the computations and errs on the safe side. Engineers are accustomed to making a similar assumption in case of concrete arches, when they determine the stresses in a section at right angle to the center line of the arch. In view of this fact, it would be well to calculate the fillet in Fig. 4 along a line about bisecting the angle, *N′ N I*, which will be found the dangerous section.

* "The Reinforced Concrete Pocket Book."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.

Discussion.*

By Messrs. Allerton S. Cushman and Samuel Tobias Wagner.

Allerton S. Cushman, Assoc. Am. Soc. C. E. (by letter).—Mr. Sabin's exposition of the present situation in regard to paint protection of steel would appear to have at least the merit of simplicity. In a few brief paragraphs he sums up all that he cares to consider of the researches, theories, and conclusions of contemporary investigators of corrosion and paint problems, and sets forth what he terms the "whole history" of this "jargon" about inhibition and stimulation, which he states never had any particular value to the consumer, and is generally used to mislead him. Mr. Cushman.

It does not seem fair or just that such a statement should be allowed to pass unchallenged, in view of the great number of engineers and others who are responsible for the care of steel structures and believe in the value of the "inhibitive" quality of protective coatings.

Mr. Sabin's review entirely ignores the electrolytic theory of the mechanism of the reactions which lead to metallic corrosion, and brushes aside all its application to paint problems, as of little or no value. Ignorance, even in the purest sense of the word, of a mass of accumulated evidence and data, is not argument, and is likely to be considered on a par with King Canute's effort to order back the advancing tide. The electrolytic explanation of corrosion, on which the inhibitive theory of paint protection is based, is now accepted by a great number of investigators and authorities in America and abroad. As the writer, to the best of his knowledge and belief, is largely re-

* This discussion (of the paper by A. H. Sabin, Assoc. M. Am. Soc. C. E., published in the November, 1913, *Proceedings*, and presented at the meeting of January 7th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Cushman.

sponsible for the "jargon" which Mr. Sabin claims has done so much to deceive consumers, he may perhaps be permitted to contribute a few paragraphs to Mr. Sabin's "whole history" of the subject under discussion. As far as the writer is aware, the first use of the words "inhibition" and "stimulation", as applied to corrosion and protection problems, occurred in his paper on "The Corrosion of Iron".* In that paper the electrolytic theory of corrosion was developed. A few excerpts from this paper will serve to illustrate the point made:

"It has long been known that rusting is inhibited and that polished iron will remain bright indefinitely in all sorts of alkaline solutions, provided they are sufficiently concentrated. This is also true of all solutions of salts of strong bases and weak acids which hydrolyze to an alkaline reaction. This fact has been eagerly seized upon by the adherents of the various theories which have been advanced, as it can be made to fit in more or less well with them all. Thus, alkalies absorb carbon dioxide, and therefore carbonic acid is prevented from carrying on its work of destruction. The added fact that fully saturated bicarbonate of soda also provides full protection to iron, even in fairly dilute solution, which would seem to be a stumbling block, has not shaken the faith of the devout believers in the carbonic acid theory.

* * * * * * * *

"Solutions of chromic acid and its soluble salts, such as the chromate and bichromate of potash, inhibit the rusting of iron immersed in them.

* * * * * * * *

"The writer has observed that if a rod or strip of bright iron or steel is immersed for a few hours in a strong (5 to 10 per cent.) solution of potassium bichromate, and is then removed and thoroughly washed, that a certain change has been produced on the surface of the metal. The surface may be thoroughly washed and wiped with a clean cloth without disturbing this new surface condition. No visible change has been effected, for the polished surfaces examined under the microscope appear to be untouched. If, however, the polished strips are immersed in water it will be found that rusting is inhibited. An ordinary untreated polished specimen of steel will show rusting in a few minutes, when immersed in the ordinary distilled water of the laboratory. Chromated specimens will stand immersion for varying lengths of time before rust appears. In some cases it is a matter of hours, in others of days or even weeks before the inhibiting effect is overcome.

* * * * * * * *

"If iron or steel is brought into contact with water, either pure or natural, iron goes into solution by replacing the hydrogen ions that are present, and if oxygen is present, the ferrous ions are oxidized with the formation of ferric hydroxide in the form of rust. Any substance which increases the concentration of the hydrogen ions in the water, such as carbonic, sulphuric, or other acids, will stimulate corrosion. On the other hand, any substance which decreases the concentration of the hydrogen ions, or in any manner prevents the interchange of

* *Proceedings*, Tenth Annual Meeting, Am. Soc. for Testing Materials, June, 1907, p. 211.

Mr. Cushman.

the electrostatic charge between the metal and the hydrogen ion, will inhibit corrosion. It follows from this that the rusting of iron is primarily a hydroxylization rather than an oxidation, and the oxygen does not directly attack the surface of the metal in the wet way or at ordinary temperatures.

* * * * * * * *

"Substances which increase the concentration of hydrogen ions, such as acids and acid salts, stimulate corrosion, while substances which increase the concentration of hydroxyl ions inhibit it. Chromic acid and its salts inhibit corrosion by producing a polarizing or dampening effect which prevents the solution of iron and the separation of hydrogen."

These citations are sufficient to show the writer's responsibility in the application of the words "inhibition" and "stimulation" to the discussion of corrosion problems. In the same paper, the practical application of the use of chromic acid and its salts as an inhibiting agent was suggested and foreshadowed in the following words:

"From what has been shown in regard to the inhibitive action of the chromates it is not improbable, since such dilute solutions prevent electrolysis and corrosion, that the addition of small quantities of bichromate to boiler waters would be highly efficacious in preventing the rapid pitting which has caused so much trouble.

* * * * * * * *

"The experiment has been made of keeping iron and steel in dilute boiling solutions of bichromate, through which a current of air is bubbled, for protracted periods, and as long as the strength of the solution was equal to or above one or two pounds of bichromate to 1 000 gallons of water no rusting has ever taken place. There would, therefore, seem to be no reason why potassium bichromate should not come into use as a boiler protective. The application of the various inhibitors in the priming coats of paints and other protective coverings has already been to some extent made use of, and it would appear that slightly soluble chromates should be theoretically the best protectives for the first application to iron and steel surfaces."

At the 1908 meeting of the American Society for Testing Materials the writer presented a paper entitled "The Inhibitive Power of Certain Pigments on the Corrosion of Iron and Steel", and in that paper the whole subject of the inhibitive theory as applied to paint protection was developed further.

Following the presentation of the writer's 1907 paper, and in the discussion of a paper by Mr. L. S. Hughes, entitled "The Deleterious Ingredients in Paints", Mr. G. W. Thompson, who is quoted by Mr. Sabin, suggested the water pigment tests as described by Mr. Sabin, and described some experiments which he had already made on this subject. Mr. Thompson, in the course of his remarks, stated that the remarkable thing which appeared in his original experiments was that some of the pigments had inhibited the oxidation, one pigment having

Mr. Cushman. caused a loss to the steel test piece of 0.37%, showing, according to Mr. Thompson's own words, a large inhibition, whereas another pigment had caused a loss of 2%, a considerable increase, as compared with the blanks which were run simultaneously. Mr. Thompson summed up by stating that he wished to make the suggestion to Mr. Cushman, Mr. Walker, and others who were investigating the corrosion of iron, that they make experiments to discover the relative electrolytic activity of various pigments in relation to iron. This suggestion was eagerly accepted by the writer, and, as Mr. Sabin describes, a committee of five chemists, from different parts of the United States, made a series of tests of the principal pigments on steel immersed in water. The results obtained by these laboratory investigations were then placed on the practical scale of experimentation by the steel test fence erected at Atlantic City by the Paint Manufacturers' Association of the United States, the inspection of which from time to time was placed under the supervision of suitable committees of the American Society for Testing Materials. As a result of these tests, which have been carried on constantly since 1908 and have been reported on annually, the best protection has been developed by the basic chromate of lead known as American vermilion. The pigments which received the five leading grade marks in these tests depend on the chromate principle of inhibition. A number of the other leading pigments depend on the basic principle of inhibition, including both the orange mineral and red leads, which have also ranked comparatively high in the tests.

In spite of this, Mr. Sabin states:

"It is obvious that in a good paint the pigment particles are enveloped in a film of oil; they do not come in contact with the iron; if they did, the paint would peel off, for no dry pigment adheres well to metal. It is as true to-day as it has been in the past that steel rusts because air and moisture act on it; and paints are used to keep air and moisture from it. They do not inhibit rusting, except as they inhibit the cause of it."

This statement would appear to show that Mr. Sabin has not comprehended the manner in which inhibitive pigments tend to protect steel when mixed with oil and painted on the surface of the metal. No intimate contact of the pigment particle and the underlying surface is in fact necessarily called for, in order that the inhibitive action should be exerted. A simple experiment can be made to prove this, by any one interested in making it. If a slightly soluble chromate, such as zinc yellow or the precipitated chromate of calcium, is mixed and ground with linseed oil, painted on a steel surface, and allowed to harden in the air in the ordinary way, and if this painted steel specimen is then immersed in water, it will be found by the gradual yellowing of the water that a small part of the chromate pigment is capable of dissolving in the water and even of being leached out of

the film. This simple experiment shows that when a chrome inhibitive paint is used for protecting iron and steel, the water of the atmosphere which condenses or rains on a surface and is absorbed by the linoxyn coating, impregnates itself to a slight degree with the chrome radical of the pigment. This solution wets the metallic surface and to some extent maintains it in a passive or non-rusting condition. Though it is true that an ideal paint film should succeed in keeping air and moisture from the underlying surface, the fact is that paint films do not exhibit ideal qualities in this respect, and all of them are more or less permeable to water and atmospheric gases. Mr. Cushman.

It is well known that for many years red lead has been used more or less successfully as a prime coating material on iron and steel, and the writer has no criticism to make of properly manufactured red leads which are properly mixed with a good vehicle and properly applied to steel. Unquestionably, it must be admitted that these red lead prime coaters yield good protection. When, however, Mr. Sabin claims that nothing more can be done, and that there is no need of further investigation of paint protection problems, the writer must take issue with him, as it can easily be shown under test that, based on the inhibitive explanation of protection, many pigments are capable of giving better protection than even the best red lead. As a matter of fact, if the use of red lead as a prime coating material has already solved the problem of protecting steel to the maximum degree, it may well be questioned why it is that so large a percentage of structural steel all over the world is suffering from rapid corrosion and deterioration. Most of the inhibitive prime coating materials must inevitably cost more than red lead, and where, as is generally the case, the first cost of application is a factor of the highest importance, red lead will probably continue to be generally used. It is the writer's belief, however, that where the first cost of application is considered in relation to the maintenance or upkeep cost of structural steel for a number of years, it is now possible, as the result of research and investigation, to design an inhibitive prime coating material which will protect steel more efficiently than the usual red lead application, and he sincerely hopes that engineers and other consumers will not remain satisfied that the last word has been said with respect to paint protection in Mr. Sabin's presentation of the present situation in regard to the painting of structural steel.

Samuel Tobias Wagner, M. Am. Soc. C. E. (by letter).—The question of the selection of a proper paint to be used on a steel structure is one of the greatest importance, and much progress has been made in the last generation toward obtaining purer materials and applying them in a more workmanlike manner. For a number of years the writer has been of the opinion that it was not fair to expect any single Mr. Wagner.

Mr. Wagner. paint to give the best results under different conditions of climate and exposure, and that generally specific conditions could be ascertained to determine which kind should be used. Throughout this discussion there were two general features which were kept prominently in the foreground; First: the materials should have a high degree of purity; Second, they should be very finely ground.

For the past 20 years the writer has looked on red lead in linseed oil as one of the best primers for structural steel, and, when mixed with some other equally pure materials, as the best final coats. The first large contract under which he used this material was in 1897 on some bridges, aggregating about 5 000 tons, on which the following specification was used:

"Red lead in oil shall be of a good bright color and very finely ground. The pigment shall contain at least 95% of red oxide of lead; no sample will be accepted that contains more than 2% of foreign matter that is vitrified, or that contains metallic lead; or that when mixed with linseed oil and drying japan without grinding, and applied in a good body to a vertical surface of iron will not dry without running or separating."

The steel on this contract was erected promptly after fabrication, and no special opportunity was given to examine the wearing qualities of the primer, but it was undoubtedly the fact that there was difficulty in filling the specification, and, when filled, the resultant paint was of such a character that it could not be applied smoothly, but either ran or was streaky. It always had to be freshly mixed in order to work it at all. The general results, however, were good.

Within the past 5 years the following experience was obtained with about 27 000 tons of structural steel bridges divided into several contracts. Nearly all the work on each structure was fabricated at one time, and the conditions on the work were that only half could be erected at that time, the other half having to be stored. Linseed oil was the primer specified on the first lot of contracts and after storing over a single winter it was found necessary to clean and re-oil a very considerable quantity of the metal. The specifications on the remaining contracts were modified by requiring that the priming coat should consist of red lead of the same specification as given above. Experience showed that after storage of more than 2 years the metal was in good condition.

The red lead required at this time was not obtained without very considerable trouble. The samples first submitted to test showed 49.08 and 56.15% of red lead; they were rejected, and the paint-maker promised to do better. Three samples were then submitted, showing 77.66, 79.56, and 83.26%, respectively, and these were rejected. These rejections caused a storm of protest, and it was stated that it was impossible to obtain a higher percentage of red lead in a mixed paint. After

Mr. Wagner.

waiting some time the general contractor changed his paint man, and the next two samples showed 96.48 and 96.68%, respectively. The next three samples showed 98.11, 98.41, and 98.63%, and it is possible that if more paint had been needed the figures would have been higher. The paint when applied to the steel had a good bright color, and had none of the streaky appearance of the red lead of 1897. The color of such a paint is noticeably different from a paint with 75% of red lead. The fact that such paint with 94% and more of true red lead, as prepared by recent methods, will not set has not been noticed by the writer, but this has been probably because he has not followed up all the details. It is an important matter, and the manufacturers are to be congratulated on effecting this improvement, as well as the fact that it has greater covering value.

The writer was one of the railroad engineers who was requested by the Committee of the American Society for Testing Materials to examine the paints applied on the Pennsylvania Railroad Bridge over the Susquehanna River at Havre de Grace, and can testify as to the good appearance of the paints which contained red lead in the finishing coats. It is also of common knowledge that in many cases, when all paint has disappeared from structural steel, it is possible still to distinguish the shop marks of red lead where that paint has been used.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILHELM HILDENBRAND, M. Am. Soc. C. E.*

DIED FEBRUARY 21ST, 1908.

Wilhelm Hildenbrand, one of the most eminent designers and constructors of suspension bridges, was born on June 1st, 1843, at Karlsruhe, in the Grand Duchy of Baden, Germany. After receiving a classical education in the Lyceum of his native town, he entered its Polytechnic School, which was then the most renowned in Germany, having among its professors, Redtenbacher, one of the founders of scientific mechanical engineering, and Sternberg, equally prominent in scientific bridge design. After he was graduated in engineering and had passed the "Staats Examen" required for Government employment, he entered its service as Engineer of Highway Construction and as Inspector of rails at the rolling mills of Westphalia and Rhenish Prussia. After a year in the service of the State, he emigrated in 1867 to America.

His first position was as Draftsman in an architect's office, but he soon found employment with the late John A. Roebling, M. Am. Soc. C. E., who was at that time planning the construction of the Brooklyn Bridge. Under Mr. Roebling's directions, Mr. Hildenbrand made the first drawings of that famous structure, in the building of which he afterward took a prominent part.

During the delay caused by the initial difficulties of this enterprise he entered, as Assistant Engineer, the service of the New York Central Railroad where he was entrusted with the architectural design for the Forty-second Street Terminal Passenger Station in New York City, and with the construction of the great arched roof over the train-shed. Both structures were built in accordance with his plans and specifications, thus laying the foundation of his reputation as an architect and engineer. The roof truss had the largest span in the world at that time, and lasted until the entire reconstruction of the station required its removal.

In 1870, when the Grand Central Station was nearly completed, the actual construction of the Brooklyn Bridge began, and Mr. Hildenbrand was engaged by Washington A. Roebling, the Chief Engineer, as Principal Assistant Engineer. During Mr. Roebling's protracted illness, which confined him to his bed or room for nearly ten years, Mr. Hildenbrand made all the scientific investigations and mathematical

* Memoir prepared by Joseph Mayer, M. Am. Soc. C. E.

calculations necessary for the structure. He also made the architectural design for the approaches and had charge of the steel superstructure, which was designed, inspected, and erected under his direction.

In 1883, after the completion of the Brooklyn Bridge, Mr. Hildenbrand opened an office in New York as Consulting Engineer and, as Chief Engineer, built many suspension bridges in the United States and Mexico. He also built a truss bridge over the Ohio River at Wheeling, W. Va., which, like most of his work, is distinguished for its pleasing appearance.

In 1885 he submitted a design in the public competition for the Washington Bridge, and was awarded the second prize. Several features of his design were incorporated in this bridge as built.

In 1894 Mr. Hildenbrand did important work for the New York Chamber of Commerce which protested against the building of a pier near the center of the North River for a railroad bridge at that time projected by the New York and New Jersey Bridge Company. The President of the United States appointed a commission of five distinguished engineers to examine the feasibility of a single-span bridge across the river. Mr. Hildenbrand submitted a design, an estimate, and an argument in favor of a suspension bridge, and appeared before the Commission as the representative of the Chamber of Commerce. The Commission reported in favor of the feasibility of a suspension and against a cantilever bridge, which was the question at issue; Mr. Hildenbrand, therefore, carried his point.

In 1895 he became Chief Engineer of the reconstruction of the Covington and Cincinnati Bridge over the Ohio River. This was his most important and difficult independent work. The bridge is the third largest suspension bridge in the world, and was built originally by Mr. John A. Roebling in 1867, but had become inadequate for the increased traffic. Mr. Hildenbrand replaced the floor and the stiffening trusses with a wider floor and new stiffening trusses without interrupting traffic, and supplemented the old cables and anchorages by additional ones. The task of distributing the load between the new and the old cables was difficult, and required accurate calculation and delicate adjustment. It was evidently accomplished successfully; the new bridge is not only an adequate but a beautiful structure, and an ornament to the two cities. It was finished in 1899, to the satisfaction of his clients.

On the completion of this bridge, Mr. Hildenbrand went to Mexico, where he constructed a light suspension bridge, of about the same span as the Cincinnati and Covington Bridge, for the transport of silver ore from the mines of the Peñoles Company at Mapimi to the terminal point of a rack railway, built on the Abt system, which Mr. Hildenbrand had constructed in 1897.

As the representative of the Abt system of inclined rack railways, Mr. Hildenbrand, in 1889-1890, constructed the track of the Pike's

Peak Railway in Colorado, reaching an elevation of 14 214 ft., and furnished its locomotives.

In 1900, in the service of John A. Roebling's Sons Company, of Trenton, N. J., he became the Chief Engineer of the contractor for building the cables of the Williamsburgh Bridge. These cables, 50% larger than those of the Brooklyn Bridge, were built by him in one-third of the time required to construct those of the Brooklyn Bridge.

In 1904, he was appointed Consulting Engineer of the Westinghouse Electric Company, of Pittsburgh, and as such he designed the overhead structures for the electrification of the passenger traffic of the New York, New Haven and Hartford Railway, near New York City. He designed other and similar structures for the Westinghouse Company, acting as Consulting Engineer for this Company until his death.

In 1877 Mr. Hildenbrand published a treatise on "The Theory and Construction of Wire Cables," which is a standard work on the subject. In 1888 he published a book on "Underground Haulage of Coal by Wire Rope," which was awarded a medal and diploma of honor at the World's Fair of Chicago in 1893.

In 1895 he published in the German language a lecture on the history of suspension bridges from the earliest to the latest times. He also published many articles in the technical press, mostly referring to suspension bridges.

As an engineer and architect, Mr. Hildenbrand always showed good taste, and gave close and painstaking attention to all details. His extreme conscientiousness, which made him hesitate to delegate minor parts of his work to subordinates, was the only obstacle to his attaining a still more brilliant success than he achieved.

He had a very interesting and original personality, and was a true friend to many who were fortunate enough to make his acquaintance. He was liberal to a fault in his assistance to many who needed it. He was an enthusiastic admirer of Wagner, played his operas on the flute, and never missed attending their performance when opportunity offered.

He was married, in 1901, to Miss Hubbard, daughter of Judge Hubbard of Covington, Ky., who survives him.

Mr. Hildenbrand was elected a Member of the American Society of Civil Engineers on February 5th, 1902.

GEORGE BROWNE POST, M. Am. Soc. C. E.*

Died November 28th, 1913.

George Browne Post, the son of Joel B. and Abby M. Post, was born in New York City, on December 15th, 1837. He was educated at Churchill's Military School at Sing Sing, N. Y., and the Scientific

* Memoir prepared by the Secretary from information supplied by George B. Post and Sons.

School of New York University, from which he was graduated as Civil Engineer with the Class of 1858.

After his graduation, and during 1858, 1859, and 1860, he studied Architecture with the late Richard M. Hunt. In the latter year he formed a partnership with a fellow-student, Mr. Charles D. Gambrill, for the practice of Architecture.

During the Civil War, Mr. Post served in the Army for 4 months in 1862 and for 4 months in 1863, as Captain in the Twenty-second Regiment, New York Volunteers. At the first battle of Fredericksburg, he acted as Volunteer Aide on the staff of General Burnside, commanding the Army of the Potomac. Mr. Post was promoted to the rank of Major, Lieutenant-Colonel, and then Colonel of the Twenty-second Regiment.

After the War he resumed his professional career, and his partnership with Mr. Gambrill was dissolved. In 1905, with his sons, William S. and J. Otis Post, he formed the firm of George B. Post and Sons.

Among the buildings designed by Mr. Post during his long career, are the Prudential Life Insurance and the Mutual Benefit Life Insurance Buildings in Newark, N. J.; the Wisconsin State Capitol; the Cleveland Trust Company Building, in Cleveland, Ohio; and the Manufacturers' Liberal Arts Buildings at the Chicago Exposition in 1893. In New York City, he designed the buildings for the College of the City of New York; the New York Produce Exchange; the New Stock Exchange; the Pulitzer Building, and the Western Union Building, in Dey Street. He also designed the Equitable Life Assurance Society Building, and several well-known private residences.

Mr. Post was a Member of the Architectural League of New York, and served as President from 1893 to 1897, inclusive. He was elected an Honorary Life Member in 1912. He was a Fellow of the American Institute of Architects, of which body he was President from 1896 to 1899, inclusive. He was a member of the New York Chapter of the American Institute of Architects, and President in 1904; of the Fine Arts Federation of New York, and President in 1898. He was a Charter Member of the National Arts Club, and President from 1898 to 1905. He was a member of the Municipal Art Society, and served as Director from 1901 to 1909. He was a member of the Council of the National Sculpture Society in 1904. He was also a Member of the National Institute of Arts and Letters, the American Academy of Arts and Letters, the Province of Quebec Association of Architects, New York Academy of Sciences, American Geographical Society, National Society of Craftsmen, Public Art League, Archæological Society of America, National Geographical Society, and the Metropolitan Museum of Art. In 1907, he was appointed Honorary Corresponding Member of the Royal Institute of British Architects. He

was elected an Associate of the National Academy of Design in 1907 and an Academician in 1908.

In 1901, Mr. Post was decorated a Chevalier de la Legion d'Honneur of France. In 1908 he received the honorary degree of Doctor of Laws from Columbia University, and in 1910, he was awarded the Gold Medal of the American Institute of Architects.

When the Tenement House Commission, known as the Gilder Commission, was appointed by the New York State Legislature, he was made a member, and, in 1902, he was appointed a member of the Board of Commissions of the St. Louis Exposition by the Governor of New York.

Mr. Post was appointed, by the Secretary of State, as a delegate to represent American Architects at large at the World's Congress of Architects held in London, and, in 1906, the Secretary of Agriculture appointed him a collaborator of the Forest Service of the United States Department of Agriculture. In 1906, President Roosevelt made him a member of the National Advisory Board on Fuels and Structural Materials, to which Board he was reappointed in 1907, 1908, and 1909. In 1909, he was appointed a member of the Bureau of Fine Arts by President Roosevelt.

He was appointed a member of the Committee of Patronage to the Eighth International Congress of Architects in 1907, and in 1908 a Member of the Permanent Committee.

He was a member of the Expert Committee to appoint a sculptor and select a design for the Lafayette Monument erected in the courtyard of the Louvre in Paris.

In 1863, Mr. Post was married to Miss Alice M. Stone, daughter of William W. Stone.

He was a member of the New York Chamber of Commerce, the New Jersey State Chamber of Commerce, Century Association, Union Club, Cosmos Club of Washington, D. C., Lawyers' Club (Charter Member), and the New York Farmers Club.

Mr. Post was elected a Member of the American Society of Civil Engineers on September 2d, 1896.

CHARLES WALKER RAYMOND, M. Am. Soc. C. E.*

Died May 3d, 1913.

Charles Walker Raymond was born at Hartford, Conn., on January 4th, 1842, and was the second son of the Reverend Robert R. and Mary A. (Pratt) Raymond. During his childhood his parents removed to Syracuse, N. Y., where he was edu-

*Memoir prepared by Dr. R. W. Raymond and Alfred Noble, Past-President, Am. Soc. C. E.

cated in the public schools and High School until 1857 when, his father becoming Professor of the English Language and Literature in the newly established Collegiate and Polytechnic Institute of Brooklyn, N. Y., the family moved to that city, and he entered that institution. As a child, he had been intensely interested in literature, but backward and awkward in mathematics. In the Polytechnic he came under the influence of Professor Richard Smith, a retired officer of the Engineer Corps (afterward President of Girard College), who was Professor of Mathematics; and, to the surprise of his family, he began to distinguish himself in that study. The strangest feature of this change was that he developed, under the inspiring touch of his brilliant teacher, not a previously unsuspected genius for mathematics, but a high ambition to conquer by persistent work the field which seemed so difficult. It was by hard work at every step that he acquired a thorough knowledge which placed him at the head of his class, and led Professor Smith to urge that he should seek an appointment as cadet in the United States Military Academy, on account of his unusual mathematical ability. In these days, when it is the fashion to educate boys mainly in the lines to which they show the earliest bent, this instance of training in directions not congenial is suggestive. We hear much about the subsequent failures of those who took the highest rank in school; but when that distinction is won, not through the easy superiority of genius, but by intense devotion to distasteful work, the man who wins it is likely to keep it always. This, at least, was the keynote of General Raymond's career, and the secret of his remarkable record of unbroken success through more than forty years of professional service.

Graduated from the Polytechnic in 1860, he was appointed in 1861 to the Military Academy, where he stood at the head of his class for four years. On his graduation, in 1865, he was immediately assigned to the Corps of Engineers with the full rank of First Lieutenant—skipping the rank of Second Lieutenant altogether. In his "furlough year" (1863), he had spent his leave of absence as an officer on the staff of Major-General Couch, by special appointment of the Secretary of War. General Couch commanded the Department of the Susquehanna, and Lieutenant Raymond's service with him in June and July covered the Campaign and Battle of Gettysburg.

From October 1st, 1865, to September 17th, 1866, he was Assistant to the Special Board of Engineers for improving the fortifications near Boston, Mass. He was then transferred to the Pacific Coast, where he served for a brief period as Assistant Engineer in the construction of the defences of Alcatraz Island, San Francisco Harbor, and then became Recorder of the Board of Engineers for the Pacific Coast. This position be held from December 28th, 1866, to March 3d, 1869, during which period he was promoted (March 1st, 1867) to be Captain

in the Corps of Engineers, and also conducted important operations at Lime Point, San Francisco Harbor, where he was Assistant Engineer for two periods—April 24th to July 7th, 1868, and November 5th, 1868, to March 3d, 1869. The work at Lime Point involved the removal of large masses of rock by heavy blasts. The interval between the two periods named was occupied with the reconstruction of Fort Stevens, at the mouth of the Columbia River, Oregon.

In 1869, after the purchase of Alaska by the United States, the American Fur Company, which had succeeded to the business of the Russian Fur Company, sent a small party up the Yukon River as far as Fort Yukon, a post of the Hudson Bay Company, to investigate the fur trade of the upper river and decide whether to establish an American station there. Captain Raymond was ordered by the Commanding General of the Department of the Pacific to accompany this party, for the purpose of making a reconnoissance of the lower Yukon, and determining astronomically the position of the meridian which constitutes the Eastern Boundary of that part of Alaska. Accompanied by one assistant and furnished with an astronomical field outfit, he joined the traders' party, which left San Francisco on April 6th, 1869, in a sailing brig, carrying on deck a small steam pinnace, and bound for Sitka, at which point his party was reinforced by one private soldier detailed for this service. Leaving Sitka on May 9th, the expedition, after many delays, reached the mouth of the Yukon, launched the little steamer, and proceeded in it up the river to Fort Yukon, a distance of, perhaps, 1,000 miles. During this trip, Captain Raymond and his assistant, relieving each other day and night, made a reconnoissance from which he afterward prepared the map of the lower Yukon which accompanied his official report and which was undoubtedly the best which had appeared up to that time. Arriving at Fort Yukon on July 31st, 1869, the traders' party soon ascertained that the fur trade of that region was not large enough to warrant the establishment of an American agency; and, as the freezing of the northern tributaries of the river was already threatening to make it too shallow for their steamer, they resolved to return at once, leaving Captain Raymond and his two companions to finish the astronomical work and make their way to the coast by themselves, but promising to hold the brig for them at the mouth of the river until a certain date. Establishing a field observatory, and observing the solar eclipse of August 7th, Captain Raymond determined the meridian boundary, proved that the British post was on American territory, and raised the American flag over it. (The post was immediately removed to the other side of the meridian.)

Leaving Fort Yukon on August 28th, his little party descended the river in a rude boat constructed by themselves—the Indians refusing to venture the voyage in their canoes at that season. This vessel con-

veyed them precariously as far as Anvik (about 500 miles from Fort Yukon and 500 miles from the sea), where it went to pieces. The remainder of the journey was characterized by hardship, exposure, and even starvation; and the explorers, exhausted and emaciated, reached the appointed rendezvous just as the brig was hoisting sail for San Francisco.*

From January 7th, 1870, to June 16th, 1871, he was Secretary of the Board of Engineers for Fortifications, etc., etc., of the United States; and from the latter date to August 23d, 1872, he commanded the Engineer Company at Willets Point, N. Y. On August 28th, 1872, he became Principal Assistant Professor of Natural and Experimental Philosophy at the U. S. Military Academy, and filled this position until February 27th, 1874, and again from August 31st, 1875, to July 1st, 1878—the interval being spent on special service in command of the U. S. Expedition to Northern Tasmania to observe the transit of Venus. From August 28th, 1878, to August 27th, 1881, he was Instructor in Practical Military Engineering, Signaling and Telegraphy, and during a part of that period served also as Superintending Engineer of Construction of the West Point Water-works, the Cadet Hospital, and the Cadet Barracks Extension. In the various positions mentioned, Captain Raymond spent more than seven years in actual service at the Academy.

During this period, he published an essay on Terrestrial Magnetism, which added to his already established reputation as a mathematician and physicist.

From August 30th, 1881, to January 13th, 1883, he commanded the Engineer Company at Willets Point. In January, 1883, he was placed in charge of river and harbor improvements, surveys, and coast defences in Massachusetts, and retained this position until February, 1886. The following condensed catalogue of his duties, compiled from the Annual Reports of the Chief of Engineers, will give some notion of their extent and variety: The protection and improvement of Boston Harbor, in which he established a practicable 28-ft. channel to the ocean; operations of similar character at the Harbors of Newburyport, Scituate, Plymouth, Provincetown, Lynn, etc., and Sandy Bay; studies and improvements of the Merrimac and Malden Rivers, and numerous other preliminary examinations, surveys, and projects. The most important of these works, from an engineering standpoint, was the improvement of Newburyport Harbor, at the mouth of the Merrimac, where Captain Raymond, through modifications of plan and method, secured, not only an increase of channel depth on the bar from 7 to 18.5 ft., but a saving in expense over the original scheme,

* Captain Raymond's official report, published as Senate Executive Document No. 12, of the Forty-second Congress, says little about the sufferings of his party, but they can be read between the lines. His health was permanently impaired.

which contemplated only a 17-ft. channel. His report on Sandy Bar (1884) contains a brilliant and novel discussion of the cross-section of a breakwater, and the anchorage capacity of a harbor, which contributed new ideas to engineering literature. On February 20th, 1883, he was promoted to be Major in the Corps of Engineers.

During his term at Boston, he served also for the greater part of 1883 and 1885 as Engineer of the First and Second Lighthouse Districts (covering the Coast of Massachusetts and Maine); and from October, 1883, to April, 1884, he superintended the removal of a wreck from Gloucester Harbor.

In February, 1886, he was ordered to take charge of levees and other improvements on the Mississippi River, from Warrenton to its delta, and from that position he was called, December 7th, 1886, to Washington, D. C., as Assistant to the Chief of Engineers. This office (which often made him temporarily Acting Chief of the Corps), he retained until January 26th, 1888, when he was appointed Engineer Commissioner of the District of Columbia—one of the three Commissioners constituting the government of the District. He served until February 1st, 1890, dealing, among other works of municipal engineering, with the difficult problem of electrical subways, concerning which his official report furnished a valuable theoretical and practical discussion.

From February 13th, 1890, to September 30th, 1902, he was (with the exception of two months in the latter year) continuously in charge of the defenses, harbor improvements, etc., at Philadelphia and in Delaware River and Bay. This work included the completion, on a novel plan and by a method designed by Major Raymond, and with great saving in both time and cost, of the now famous Delaware Breakwater which, disproving all the sinister prophecies of its early critics, stands unmoved after ten years of practical trial, a monument to his courage and originality as an engineer.*

The following sketch of his work in the Philadelphia District is compiled from the Annual Reports of the Chief of Engineers, from 1890 to 1902, inclusive.

For the first eight years of his tour of duty, the project for the Delaware River up to Philadelphia provided for a depth of 26 ft. at mean low water, and from Philadelphia up to Trenton for a depth of 12 ft. In the upper river above Philadelphia the principal obstruction to the 12-ft. channel was Kinkora Bar. At this point a dike had been built before he took charge. The natural depth was 7.5 ft. He suc-

* This work is described in the "Final Report of Lt.-Col. Charles W. Raymond, Corps of Engineers, Upon the Improvement of Delaware Breakwater," Reports of the Chief of Engineers, U. S. Army, 1899, Part II, p. 1346 *ff.* It is a significant circumstance that when Major Raymond took charge of it, in 1891, a plan had been already approved, in 1890, by the U. S. Board of Engineers, which he was expected to carry out; so that, in order to put his own conception into effect, he had to move that Board to recede from its own recently adopted scheme and adopt a new one, comprising features not supported by engineering precedents.

ceeded in obtaining greater depths, but the work done was not permanent in effect.

During these eight years he made annual examinations of the shoals in the river at Smith's Island Bar, Mifflin Bar, Schooner Ledge. Five Mile Bar, Bulkhead Shoal, Cherry Island Flats, Dan Baker Shoal, and other places. At most of the shoals and bars he obtained the projected depth of 26 ft.

Five Mile Bar is just above Philadelphia. It was improved by a dike and by dredging. Smith's Island was completely removed by dredging. Petty Island was partly removed. Ledges of rock were removed by blasting and dredging. Mifflin Bar was improved by dike construction and dredging. Schooner Ledge was deepened by rock removal. Cherry Island Flats was deepened by dredging. Bulkhead Shoal was deepened by the construction of Finn's Point dike. Dan Baker Shoal was originally planned to be deepened by a long dike from Reedy Island to Liston's Point, but this plan was later abandoned by reason of a decrease in the cost of dredging, and the channel was deepened by dredging alone.

In 1898 the condition of the river was good except at a few points. At Dan Baker Shoal the least depth was 16.5 ft. at low tide. In that year, a survey was made for the purpose of determining the possibility of still greater improvement, and on the basis of this survey a Board of Officers recommended a project for a channel from Philadelphia to Delaware Bay, 30 ft. deep and 600 ft. wide. Major Raymond started this improvement, deepening the channel to 30 ft. as rapidly as funds were made available. As the Delaware River is a silt-bearing stream which tends to obliterate artificial channels not placed exactly where Nature would place them, the improvement involves continual maintenance work. Even ten years after he had left the district, the work was still in progress, and it will always be continued as long as the river is used for commerce.

One of the great problems involved in this undertaking consisted in finding suitable places to dispose of dredged material. Major Raymond combined a dike for the improvement of Dan Baker Shoal with a plan for a disposal basin, making in mid-stream an artificial island about three miles long. The bulkhead was built, and the basin was so large that it has received dredged material for many years and is not yet full. This island is locally known as Raymond Island.*

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*The following minute of the Executive Council of the Philadelphia Board of Trade shows the estimate placed on his work by that body:

"*To the President and Members of the Executive Council of the Board of Trade:*

"GENTLEMEN:

"Your Committee on the Improvement of the Harbor and Delaware and Schuylkill Rivers respectfully reports:

"That it has noted with regret the death of General C. W. Raymond, U. S. A., and in view of the services rendered by him to the city of Philadelphia during the twelve years he was in charge of this district as Engineer Officer, presents for your adoption the following minute:

"The Executive Council of the Philadelphia Board of Trade records this minute

In May, 1898, he became Lieutenant-Colonel, Corps of Engineers, and in May, 1901, a member of the U. S. Board of Engineers, of which, as a young Captain, he had been the Secretary, 31 years before. He remained a member of this Board until his retirement from active service in 1904.

During this period he completed the great work of an analytical index of the Reports of the Chief of Engineers from 1866 to 1900, inclusive. This book, in three volumes, is a model of intelligent and convenient classification, as well as comprehensiveness and accuracy.

It was in these later years of his life that some of his most important work was performed. As already observed, he served as member on many Boards and Commissions, reporting on fortifications, rivers, harbors of refuge, etc. Among these, the most noteworthy were: The Board to decide between San Pedro and Santa Monica Bay, for the location of a deep-water harbor on the coast of California; the Board to determine the maximum span for a suspension bridge, and especially for such a bridge over the Hudson River at New York; and the Board to determine the route and cost of a deep waterway from the Great Lakes to the Atlantic.

Of the two latter, he was the President. The report of the Bridge Board contained an analytical discussion of the Theory of Suspension Bridges, mainly prepared by him, a translation of which was subsequently used in Europe as a textbook of instruction. The Deep Waterways Board made careful surveys for a ship canal from Lake Erie to deep water on the Hudson River, below Albany, and, with the aid of maps furnished by the Engineer Corps of the Army for the chan-

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upon the death of General C. W. Raymond, U. S. A., which took place after a lingering illness May 3, 1913.

"The city of Philadelphia owes a debt of gratitude to General Raymond for his intelligent and untiring efforts for the improvement of the navigation of the Delaware and Schuylkill Rivers and the removal of Smith's and Windmill Islands, which made possible a widened Delaware Avenue, and the building of wharves of such dimensions as to accommodate modern vessels.

"The work on the improvement of the harbor was prosecuted under his direction from the time the title to the islands was vested in the United States, under condemnation proceedings, May 29, 1890, until the completion of the readjustment of the harbor conditions, January 10, 1898, and during all that time he was responsive to every suggestion of the commercial and maritime interests looking to the early and successful termination of the work which has inured so greatly to the advantage of the city and port.

"It was under General Raymond (then Major) that the initial dredging for a 30-ft. channel, as provided by Act of Congress, March 3, 1899, took place, and was continued until he was relieved from duty at Philadelphia on July 20, 1901. He designed and practically carried to completion the great constructions belonging to the National Harbor of Refuge at the entrance of Delaware Bay.

"General Raymond's pre-eminent qualifications as an engineer were universally acknowledged, and his appointment as Chairman of the Commission to plan and construct the tunnel approaches to the New York Terminal of the Pennsylvania R. R. Co., was a practical recognition of his high standing in his profession; and the success of the undertaking furnished additional proof of his unrivaled technical skill and executive ability.

"The members of the Executive Council in adopting this minute, desire to express their high appreciation of the services of incalculable value rendered the port of Philadelphia during the twelve years General Raymond had charge of this district, and at the same time to tender his family their sympathy in the loss sustained by them.

"On motion, this minute was adopted by a rising vote and the Secretary instructed to send a certified copy to the family of General Raymond."

nels above Lake Erie, prepared detailed estimates for the whole line from the head of lake navigation. The report of the Board, which was the joint work of its members and assistants, was universally recognized as the most complete and thorough of its kind in the literature of engineering.

In January, 1904, he became Colonel, Corps of Engineers; and on June 11th of the same year, he was retired, at his own request, with the rank of Brigadier-General, U. S. A., after more than forty years of consecutive active service. The Government, however, still required him as one of its representatives in the International Congress of Internal Waterways, of which he was, from 1902 until his death in 1913, a member of the Council, Chairman of the American Section, an attendant at the meetings of the Congress, and a contributor to its Proceedings. The latest of these meetings was held in 1912 in the United States, and General Raymond would have been its presiding officer, had he been able to be present.

One of the most memorable labors of his life was the last. Already several years before his retirement, he had been permitted by the War Department, at the urgent request of the Pennsylvania Railroad Company, to act as Chairman of the Board of Engineers created by that Company to supervise the design and construction of the vast improvement contemplated by it in and around New York City, including the tunnels under the Hudson River, the East River, and the Borough of Manhattan; the great Pennsylvania Terminal in New York; and the terminals and yards on Long Island and in New Jersey. General Raymond was not an idle member of that supreme body; all important features of the plans and specifications were passed on by the Board, to the work of which he gave unremitting attention for more than eight years. The many studies and investigations carried out by him during that period were fruitful in determining the final design. Among the most important results of his special labors was the discovery of a minute diurnal rise and fall of the tunnel tube, under the influence of the tides. His recommendation that the tunnel should be left free to move, without any attachment of piles to resist either rise or fall, was adopted by the management of the Company, and has been vindicated thus far by experience. The great tube now adjusts itself freely to the changes in pressure of the material surrounding it, and its minute but practically irresistible movements do not affect its stability, in which, by reason of its immense weight, the effect of passing trains is likewise a negligible factor. The name of General Raymond worthily stands at the head of the list of engineers, on the great memorial tablet at the portal of the Pennsylvania Railroad Station in Seventh Avenue, New York City.

These labors were performed under difficulties which might well have discouraged a less intrepid spirit. General Raymond had lost

almost entirely, some thirty years before, possibly as the result of the hardships of his explorations in Alaska, the use of one eye; and, about 1900, the formation of a cataract in the remaining eye threatened him with entire blindness. Under this increasing disability (which finally reached such a point that he had to be personally led to and through the tunnels, and could examine drawings only with the aid of a strong glass, magnifying one spot at a time), he continued the active and efficient discharge of his duty as head of the directing Board of Engineers until, in 1910, its work was practically done, and his office in New York was closed. Even after that, in his seaside cottage near Atlantic Highlands, N. J., he dictated, at the request of the Pennsylvania Railroad Company, and under great difficulties of ever-growing blindness, a report of the work of his Board, which has been published by the Company and constitutes a memorable contribution to the literature of that branch of engineering.

Meanwhile, many successful operations (each partly, but not decisively, successful) had been performed on his eye, in which the obscuring film repeatedly gathered. In October, 1912, after months of total blindness, he went to Washington once more for another such operation, with more than usual expectation of permanent relief. The surgeons at Washington, however, discovered another and previously unsuspected trouble—an internal malignant tumor which forbade the expected operation and was itself beyond cure. There was nothing left on earth to him but hopeless darkness and cruel pain, which he bore with characteristic fortitude and patience until his death on May 3d, 1913. Conscious and serene to almost the last moment, he crowned with a heroic death a long, active, useful, and distinguished life, graduating at the end, as he had graduated at the Military Academy nearly half a century before, "at the head of his class." The Alumni of West Point may well be proud of his stainless and illustrious record.

Charles Walker Raymond was elected a Member of the American Society of Civil Engineers on June 1st, 1892.

BAIRD SNYDER, Jr., M. Am. Soc. C. E.*

DIED JULY 9TH, 1913.

Baird Snyder, Jr., was born at Pottsville, Pa., on November 21st, 1868. He was the third son of Baird and Edith Morris Snyder. The mother, now deceased, was a great-granddaughter of Robert Morris, so distinguished in the days of the American Revolution and a signer of the Declaration of Independence. His father, Baird Snyder, was the son of George Washington Snyder, one of the early pioneers of

*Memoir prepared by Mr. Frank A. Hill, Gen. Mgr., Maderia, Hill & Co., Inc., Pottsville, Pa.

the Anthracite Region, and the three generations have been prominent in the development and progress of mining in the district.

On his graduation from the Pottsville High School in 1885, Mr. Snyder entered the service of the Pennsylvania Railroad Company as Clerk in the office of the Chief Engineer.

In 1888 he joined the Engineer Corps of the Philadelphia and Reading Coal and Iron Company, and began the engineering career of which his friends and associates are so justly proud and which broadened in experience and grew in accomplishment until his valued life was suddenly closed.

In 1893, the late Joseph S. Harris, while President of the Lehigh Coal and Navigation Company, recognizing Mr. Snyder's ability, appointed him Assistant Superintendent of the mining operations of that Company at Lansford, Pa. Three years later he was made General Manager of the Company, and remained in this position until January, 1912.

The property which came under his management is one of the oldest and best in the Anthracite Region. The various adverse conditions which arise in the experience of the anthracite mine manager were all present during Mr. Snyder's administration at Lansford. The many difficulties of mining, pumping, ventilation, labor, fighting fire under ground, and countless others, were met with patience and self-reliance and were successfully overcome. Among Mr. Snyder's most marked qualifications for his position was his ability to handle men. No man had more loyal subordinates to aid him, and no men ever had a mort intrepid leader.

After nineteen years of active and aggressive work at Lansford, he resigned from his position with substantial expressions of confidence and regard from his Board of Directors and with the heartfelt regret of his subordinates.

On leaving the Lehigh Coal and Navigation Company, Mr. Snyder, as President and General Manager, organized the Locust Mountain Coal Company, for the purpose of operating an undeveloped area of coal land owned by the Stephen Girard Estate. He went into this enterprise with his characteristic fearlessness and vigor. He had completed his financial and working plans, and was in the midst of their early development with every confidence of success, when death came.

He had met and conquered the many engineering difficulties that had come to him in his old work, and was looking forward with pleasure to solving the new and less complicated questions which would meet him in opening an undeveloped territory of his own selection, but on July 9th, 1913, from injuries received in an automobile accident on the preceding day at Wapwallopen, Pa., this strong and virile life, filled with the promise of many more successful years, was cut off

"in the twinkling of an eye", leaving to his family and his many friends and professional associates only the memory of an active and well spent life, full of achievement; the record of an honest man, a good citizen, an able engineer—a record creditable to the membership of any engineering society.

Mr. Snyder was a man of splendid physique, of fine mentality, a reader, and a student. Positive, even to brusqueness, without fear, and a natural leader, he was also a devoted son, a loving husband, and a kind father. He is survived by his widow, Jennie Craig Romig Snyder, and two sons, Baird Snyder, 3d, and Robert Morris Snyder.

Mr. Snyder was elected a Member of the American Society of Civil Engineers on March 2d, 1904.

JAMES DYNAN NEWTON, Assoc. Am. Soc. C. E.*

Died August 8th, 1912.

James Dynan Newton was born at Oswego, N. Y., on April 17th, 1871. He entered Holy Cross College, Worcester, Mass., in 1887, and was graduated in 1891 with the degree of Bachelor of Arts. He taught sciences in the Roman Catholic High School, Philadelphia, for one year and then entered Sibley College, Cornell University, and received the degree of Mechanical Engineer in 1895. In the same year he was granted a Master's Degree from Holy Cross College.

After his graduation from Cornell he served as Special Apprentice in the New York Central Railroad shops at Oswego for five months. In December, 1895, after passing the examinations, he was appointed Cadet Engineer in the United States Revenue Cutter Service, and in 1896 he was promoted to Third Lieutenant of Engineers. He served in this capacity till 1902, when he was retired for disability incurred in the line of duty. He was in the Revenue Cutter Service during the Spanish-American War, and for a part of this time was Acting Chief Engineer of the U. S. S. *Hamilton*.

He was sent to the Marine Hospital at Fort Stanton, N. Mex., for treatment, and after leaving there was employed on mining development in New Mexico, practiced engineering independently, and, in 1904, was employed as Chainman and Rodman on the Atchison, Topeka, and Santa Fé Railroad. In August, 1905, he accepted the position of Assistant Professor of Civil Engineering in the University of Kansas.

The writer was in charge of the Department of Mechanics during the time of Professor Newton's service in the University of Kansas,

* Memoir prepared by Herbert A. Rice, Assoc. M. Am. Soc. C. E.

and, as a part of his teaching was in this Department, it gave good opportunity for observing his work.

In the Engineering Profession, no doubt, one of the greatest virtues is that of hard, conscientious labor, and this Professor Newton possessed in a marked degree. Entering a new field, he at once saw the necessity of careful preparation. He not only prepared himself for the work in hand, but spent many hours in the study of related subjects. His broad college training and his practical experience enabled him to present his subjects in an interesting and practical manner, and the writer always found his students enthusiastic in his praise.

At the end of his fourth year at the University of Kansas, he was elected Dean of the School of Engineering of Loyola University, Chicago, Ill. Here, his great capacity for labor stood him in hand. The Engineering School which had not been established, had to be organized, courses laid out, catalogue prepared, equipment purchased for a new building then in process of construction, and instructors engaged. This entire work fell to Professor Newton. Success attended his efforts, and, at the time of his death, the school was thoroughly organized and had enrolled many students in the new Department.

Professor Newton was an enthusiastic worker in the Roman Catholic Church, being one of the leading soloists in the church choir while in Lawrence.

He was married in 1906 to Miss Minnie Medaris, of Kansas City, who, with one child, survives him. They reside at Lawrence, Kans.

Professor Newton was elected an Associate of the American Society of Civil Engineers on September 5th, 1911.

ORLOFF LAKE, Jun. Am. Soc. C. E.*

Died October 21st, 1913.

Orloff Lake, the eldest son of Duff G. and Mary Ida (Woods) Lake, was born at Baltimore, Md., on April 19th, 1883.

He was graduated from Tulane University with the degree of Bachelor of Engineering in 1905, and immediately thereafter entered the service of the Philadelphia and Reading Railway as Rodman, which position he held until April, 1906, when he resigned to accept the position of Transitman with the New York Central and Hudson River Railroad. He was soon promoted to Assistant Engineer, and had charge of office and field work on construction at Brewster; he also made surveys for

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

the relocation of the New Jersey Shore Line from Weehawken to Shadyside.

In January, 1908, Mr. Lake left the New York Central and joined the engineering forces of the writer as Assistant Engineer on general work in and near New Orleans. He remained in this employ until the summer of 1909 when he temporarily withdrew from the engineering field and transferred his activities for two years to the manufacturing business conducted by his father and uncle.

His love for his profession, however, led him to accept employment, in June, 1911, with the Barrett Manufacturing Company, and he was especially engaged in the Tarvia Department to co-operate with county and other engineers in the South who were devoting their time and talents to highway construction. Mr. Lake's education and training especially qualified him for this work, in which he continued with marked success until his sudden and untimely death.

Mr. Lake was a young man of exceptional attainments and brilliant promise. He was a tireless student and a most industrious worker, whose physical strength was without doubt often overtaxed by the tasks which he set for himself to perform. His exceeding modesty, amounting almost to diffidence, no doubt operated in retarding his professional progress during his brief life. His high ideals, sterling good qualities, and quiet forcefulness, however, were such that, as acquaintance with him ripened, confidence in and affection for him grew strong. In his death, the Profession and the Society have lost a most promising member; and in the zone of his acquaintance there is a void which may not easily be filled.

Mr. Lake was elected a Junior of the American Society of Civil Engineers on December 1st, 1908.

HENRY A. RICHMOND, F. Am. Soc. C. E.*

Died May 10th, 1913.

Henry A. Richmond was born in Syracuse, N. Y., on August 3d, 1840. He was the second son of Dean Richmond who, at the time of his death in 1866, was President of the New York Central Railroad Company.

Mr. Richmond was engaged in the grain business, as a commission merchant, at Buffalo, N. Y., and was well known among shipping circles on the Great Lakes. Later, he was connected with a lithographing business.

*Memoir prepared by the Secretary, from information supplied by C. M. Morse, M. Am. Soc. C. E.

After his retirement from active business, he became prominent in public affairs in Buffalo, as an "old-line Democrat". He was President of the Buffalo School Association, and was appointed by the late Grover Cleveland, then Governor of New York, as a Member of the first Civil Service Commission of New York State.

Mr. Richmond never married. He was a "wide traveler and a devotee of art and literature", and was liked and respected by all who knew him. He died at Los Angeles, Cal., on May 10th, 1913, and was buried in the Richmond Mausoleum at Batavia, N. Y.

Mr. Richmond was elected a Fellow of the American Society of Civil Engineers on July 7th, 1870.

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"**THE DIVERSION OF IRRIGATING WATER FROM ARIZONA STREAMS.**" A. L. HARRIS. (To be presented Feb. 18th, 1914.)

"**STEEL STRESSES IN FLAT SLABS.**" H. T. EDDY, ESQ.

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William P. Morse

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OF THE

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OF

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(INSTITUTED 1852)

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

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MINUTES OF MEETINGS OF THE SOCIETY

SIXTY-FIRST ANNUAL MEETING*

January 21st, 1914.—The meeting was called to order at 10 A. M.; President George F. Swain in the chair; Charles Warren Hunt, Secretary; and present, also, about 600 members.

Messrs. A. H. Van Cleve, J. H. Granbery, Asa E. Phillips, Frank G. Wolfe, Frederick S. Stow and Merritt H. Smith were appointed Tellers to canvass the Ballot for Officers for the ensuing year.

The Annual Report of the Board of Direction, and the Annual Reports of the Secretary and of the Treasurer†, for the year ending December 31st, 1913, were presented and accepted.

* A full report of the Sixty-first Annual Meeting is printed on pages 78 to 117 of this number of *Proceedings*.

† For these reports, see pages 11 to 22 of *Proceedings* for January, 1914 (Vol. XL).

The Committee on proposed amendments to the Constitution presented a report on a proposed amendment to Article VII, and M. T. Endicott, Past-President, Am. Soc. C. E., Chairman of that Committee, moved that this amendment be sent out to letter-ballot without change.

The motion was seconded and carried.

This amendment is as follows:

"Amend Article VII—Nomination and Election of Officers—as follows:

"Strike out Section 1, and substitute the following:"

"The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year."

Strike out the first paragraph of Section 2, and substitute the following:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, shall be appointed by the meeting to serve for two years. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve Districts; and these, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society."

Strike out the word "and" in the fifteenth line of the third paragraph of Section 2, and after the figure "7" add: "8, 9, 10, 11, 12, and 13."

A proposed amendment to Articles V and VI was then considered, and, after discussion, the chair ruled that this amendment and another proposed amendment to Article VII, must be sent out to letter-ballot of the Society as amended by the Committee.

These amendments as amended by the Committee are as follows:

"Amend Article VII—Nomination and Election of Officers—as follows:

"Strike out Section 1, and substitute the following:"

"1.—The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year."

Section 2.—Strike out Paragraphs 1 and 2, and substitute the following:

"2.—Seven corporate members, not officers of the Society, shall be elected annually by the members in the respective districts to serve for two years. They shall be elected so as to provide with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve districts and these, with the five living last past Presidents of the Society, shall be a committee to nominate officers for the Society.

The manner of election shall be as follows:

Directly after the first day of October of each year, there shall be mailed to each corporate member in each district where an election is to be held a notice with form and envelope for voting, requesting him to suggest the name of one corporate member (not an officer of the Society) from his district as a candidate for nomination for member of the nominating committee; these suggestions to be made to the Board of Direction by a preliminary letter-ballot to be received by the Board at a meeting to be held about the first of November, and opened at said meeting, and counted under the direction of the Board. The polls shall be closed at noon on the day of said meeting and the time of closing the polls shall be stated in said notice. At least thirty days before the annual meeting there shall be mailed to every corporate member, a second notice with form for voting, said notice to state the time of closing of the polls on the final ballot, and to contain as nominees for the nominating committee from each voting district the names and residences of the two persons receiving the highest number of votes in each district, and such others as may have received the same number of votes as one of these in order of their standing, and the number of votes cast for each as the result of the preliminary ballot. On the final ballot the polls shall close at noon on the day preceding the annual meeting, and the ballots shall be canvassed under the direction of the Board. The members receiving the highest number of votes in their respective districts on the final ballot shall be declared elected members of the nominating committee. Announcement of elections by this ballot shall be made at the annual meeting. In case of a tie vote in any district the annual meeting shall elect the member of the nominating committee from among the persons so tied. Vacancies in the Nominating Committee may be filled by the Board of Direction."

Paragraphs 3 and 4 of Section 2 of Article VII shall constitute a new section numbered 3, and for the words "This Committee" in the first line of Paragraph 3 substitute the words "The Nominating Committee"; and in line fifteen of the same paragraph strike out the word "and," and after the figure "7" add: "8, 9, 10, 11, 12 and 13."

Sections 3, 4, 5, 6, and 7 of Article VII shall be numbered Sections 4, 5, 6, 7, and 8, respectively.

In the second sentence of present Section 5 (new Section 6) of Article VII, insert after the word "nominated" the words "for Officers of the Society," so the said sentence shall read "This ballot shall

NOTE.—The word "district", in the 16th line of the 3d paragraph of Section 2, was not in the Committee's amendment. It has been added since the meeting by order of Hunter McDonald, President, Am. Soc. C. E. (*Secretary.*)

include the names and residences of all persons nominated for Officers of the Society in accordance with this Article, their grades of membership, and, in case of nominees for Directors, the number of the district in which they reside."

"Amend Articles V and VI as follows:

"*Article V, Section 1.*—Strike out the first sentence and substitute the following:"

"The officers of the Society shall be a President, four Vice-Presidents, eighteen Directors, a Secretary, and a Treasurer. These officers, except the Secretary, together with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction in which the government of the Society shall be vested, and who shall be the Trustees as provided for by the laws under which the Society is organized."

Article V, Section 2.—First Paragraph. In first sentence strike out ", Secretary", so that the sentence shall read as follows:

"The terms of office of the President and Treasurer shall be one year; of the Vice-Presidents, two years; and of the Directors, three years."

Second Paragraph. In first line insert after the word "Officer" the words "elected by the Society" so that the paragraph shall read:

"The term of each officer elected by the Society shall begin at the close of the Annual Meeting at which such officer is elected, and shall continue for the period above named or until a successor is duly elected."

Article VI, Section 4.—Strike out the first two paragraphs of the section and substitute therefor the following:

"The Secretary shall be a Corporate Member of the Society. He shall be elected annually by letter-ballot of the Board of Direction, which shall be ordered not less than 20 days after the Annual Meeting.

The ballots shall be returned to tellers, appointed by the President and canvassed at a meeting of the Board. A majority of the whole Board shall be required to elect.

The Secretary shall hold office until his successor is elected, shall devote his whole time to his duties, shall be under the direction of the President and the Board of Direction, and shall be the executive officer of the Society."

Article VI, Section 6.—Strike out this Section and substitute therefor the following:

"6.—The Secretary and Treasurer shall be paid salaries to be determined by a majority of the Board of Direction, this vote to be taken by letter-ballot to be canvassed by the Board of Direction. All other salaries shall be fixed, from time to time, by the Board of Direction."

The following were appointed members of the Nominating Committee to serve two years:

LEWIS D. RIGHTS	*Representing*	*District*	*No.*	*1*
JOSEPH R. WORCESTER	"	"	"	*2*
FREDERICK E. TURNEAURE	"	"	"	*3*
THOMAS EARLE	"	"	"	*4*
JOHN W. WOERMANN	"	"	"	*5*
THOMAS U. TAYLOR	"	"	"	*6*
MILO S. KETCHUM	"	"	"	*7*

Frederic P. Stearns, Past-President, Am. Soc. C. E., Chairman of the Special Committee on Valuation of Public Utilities, presented by title a Progress Report of that Committee, and offered the following motion:

"*Resolved,* That the Progress Report of this Committee, together with all discussion thereon up to September 1st, 1914, or to such later date as the Board of Direction may fix, be referred back to the Committee for further consideration; and that in the meantime the Board of Direction be requested to assign a date for the written and oral discussion on the subject."

The motion, being duly seconded, was carried.

It was moved, duly seconded, and carried that the Progress Report of the Special Committee on Valuation be not printed in *Proceedings*.

It was moved, duly seconded, and carried that the thanks of the Society be extended to the Special Committee on Valuation of Public Utilities for the very large amount of valuable work it has done on its report.

W. W. Crosby, M. Am. Soc. C. E., Chairman of the Special Committee on Bituminous Materials for Road Construction, presented by title the Report of that Committee, and offered the following motion:

"*Resolved,* That the Progress Report of this Committee together with all discussion thereon up to September 1st, 1914, or to such later date as the Board of Direction may fix, be referred back to the Committee for further consideration; and that in the meantime the Board of Direction be requested to assign a date for the written and oral discussion on the subject."

The motion, being duly seconded, was carried.

It was moved that the Special Committee on Bituminous Materials for Road Construction be continued for the present, but be directed to present a final report at the next Annual Meeting.

The motion was duly seconded, but was not carried.

It was moved, seconded, and carried that a vote of thanks be extended to the Special Committee on Bituminous Materials for Road Construction for its work in preparing its report.

Dugald C. Jackson, M. Am. Soc. C. E., presented the Report of the Special Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers, and offered the following motion:

"*Resolved,* That the Progress Report of this Committee, together with all discussion thereon up to September 1st, 1914, or to such later date as the Board of Direction may fix, be referred back to the Committee for further consideration; and that in the meantime the Board of Direction be requested to assign a date for the written and oral discussion on the subject."

The motion, being duly seconded, was carried.

A. L. Bowman, M. Am. Soc. C. E., Chairman of the Special Committee on Columns and Struts, presented a Progress Report of that Committee.*

The report was accepted and the Committee continued.

Robert A. Cummings, M. Am. Soc. C. E., Chairman of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., presented a Progress Report of that Committee.†

The report was accepted and the Committee continued.

Richard L. Humphrey, M. Am. Soc. C. E., Secretary of the Special Committee on Concrete and Reinforced Concrete, presented a Progress Report of that Committee.‡

The report was accepted and the Committee continued.

It was moved, duly seconded, and carried that, in compliance with the recommendation in the report of the Special Committee on Concrete and Reinforced Concrete, that the Progress Report of that Committee presented at the Annual Meeting of January 15th, 1913, be printed in *Transactions*.

Desmond FitzGerald, Past-President, Am. Soc. C. E., Chairman of the Special Committee on Engineering Education, reported informally for that Committee.

The Committee was continued.

On motion, duly seconded, the thanks of the Society were given to the Special Committee on Engineering Education.

The Secretary announced that the Prizes, for the year ending July, 1913, had been awarded by the Board of Direction, as follows:

The Norman Medal to Paper No. 1235, entitled "Air Resistances to Trains in Tube Tunnels", by J. V. Davies, M. Am. Soc. C. E.

The Thomas Fitch Rowland Prize to Paper No. 1231, entitled "The Laramie-Poudre Tunnel", by Burgis G. Coy, Assoc. M. Am. Soc. C. E.

The J. James R. Croes Medal to Paper No. 1222, entitled "The Problem of the Lower West Side Manhattan Water-Front of the Port of New York", by B. F. Cresson, Jr., M. Am. Soc. C. E.

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* See page 103.
† See page 104.
‡ See page 105.

The James Laurie Prize to Paper No. 1218, entitled "Construction of the Morena Rock Fill Dam, San Diego County, California", by M. M. O'Shaughnessy, M. Am. Soc. C. E.

The Secretary reported that the Board of Direction had appointed a Special Committee to Report on Stresses in Railroad Track, consisting of A. N. Talbot, Chairman, A. S. Baldwin, J. B. Berry, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, E. Gerber, Robert W. Hunt, George W. Kittredge, William McNab, G. J. Ray, F. E. Turneaure, and J. E. Willoughby.

The Secretary read a letter* from Augustus Smith, M. Am. Soc. C. E., relative to the appointment of a Board of Arbitration for adjudicating disputes arising in engineering construction, and reported that the Board of Direction had adopted the following:

"The Board of Direction of the American Society of Civil Engineers has considered the communication of Mr. Augustus Smith concerning the appointment of an Arbitration Board, and has extensively canvassed the subject with members of the Society in various localities, some of whom have had considerable experience in arbitration matters.

"After considering the many views presented, the Board is of the opinion that arbitration of honest differences is to be recommended in preference to legal process as much as possible.

"That, however, an Arbitration Board as proposed by Mr. Smith would not be practical at the present time, nor until arbitration is more extensively practised, because of the difficulty of devising a satisfactory method of selecting competent men to membership of such a Board; further, because of the impracticability of so constituting the Board of Arbitration that it may have experts in all lines so distributed that men appointed on any given case need not be drawn from too great a distance, and further, because of the difficulty of assigning men from such a Board to the satisfaction of the disputants who may prefer persons better known to themselves."

The Secretary read a communication† from W. C. Sawyer, Assoc. M. Am. Soc. C. E., Temporary Secretary of the Los Angeles Association of Members of the American Society of Civil Engineers, in reference to the name of local associations of members of the Society, and the dues of members of such associations.

On motion, duly seconded, Mr. Sawyer's letter was referred to the Board of Direction, with power.

C. H. Higgins, M. Am. Soc. C. E., introduced the following:

"*Resolved,* That the American Society of Civil Engineers believes it to be in the public interest that the practice of engineering be regulated by statute."

On motion, duly seconded, the resolution was laid on the table.

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* See page 109.
† See page 112.

The Secretary presented the Report of the Tellers appointed to canvass the Ballot for Officers for the ensuing year.

The President announced the election of the following officers:

President, to serve one year:

Hunter McDonald, Nashville, Tenn.

Vice-Presidents, to serve two years:

Charles F. Loweth, Chicago, Ill.
Gardner S. Williams, Ann Arbor, Mich.

Treasurer, to serve one year:

John F. Wallace, New York City

Directors, to serve three years:

George W. Fuller, New York City
Arthur S. Tuttle, New York City
Charles H. Keefer, Ottawa, Ont., Canada
Mortimer E. Cooley, Ann Arbor, Mich.
Eugene E. Haskell, Ithaca, N. Y.
Richard Montfort, Louisville, Ky.

Mr. Ockerson and Mr. Endicott conducted Mr. McDonald, the President-elect, to the chair.

Mr. McDonald addressed the meeting briefly.

Adjourned.

SPECIAL MEETINGS FOR TOPICAL DISCUSSION ON ROAD CONSTRUCTION AND MAINTENANCE

January 23d, 1914.—The first special meeting for topical discussion on "Road Construction and Maintenance" was called to order at 10.15 A. M.; President Hunter McDonald in the chair; Arthur H. Blanchard, M. Am. Soc. C. E., acting as Secretary; and present, also, about 225 members and guests.

The discussion on the first topic, "Engineering Organizations for Highway Work", was opened by Willis Whited, M. Am. Soc. C. E., on the first sub-division topic, "State Highway Engineering Organizations". W. W. Crosby, M. Am. Soc. C. E., took the chair. William H. Connell, Assoc. M. Am. Soc. C. E., opened the discussion on the second sub-division topic, "Municipal Highway Engineering Organizations". These topics were discussed by Messrs. Charles J. Bennett, Arthur W. Dean, H. W. Durham, R. A. Meeker, Paul D. Sargent, W. W. Crosby, John C. Trautwine, Jr., E. W. James, William de H. Washington, William Goldsmith, A. T. Rhodes, I. W. McConnell, and G. A. Ricker.

Adjourned.

January 23d, 1914.—The second special meeting was called to order at 2.30 P. M.; W. W. Crosby, M. Am. Soc. C. E., in the chair; Arthur H. Blanchard, M. Am. Soc. C. E., acting as Secretary; and present, also, about 250 members and guests.

The discussion prepared by Paul E. Green, M. Am. Soc. C. E., on the topic, "Factors Limiting the Selection of Materials and of Methods in Highway Construction" was read by the Secretary. The subject was discussed further by Messrs. Samuel Whinery, George W. Tillson, Mark Brooke, W. W. Crosby, E. H. Thomes, R. A. Meeker, R. E. Beaty, W. M. Kinney, A. T. Rhodes, William de H. Washington, William Goldsmith, Paul D. Sargent, George P. Hemstreet, G. A. Ricker, D. B. Goodsell, H. W. Durham, and Alexander Blair.

Adjourned.

January 24th, 1914.—The third special meeting was called to order at 10.30 A. M.; W. W. Crosby, M. Am. Soc. C. E., in the chair; Arthur H. Blanchard, M. Am. Soc. C. E., acting as Secretary; and present, also, about 175 members and guests.

The opening discussion on the third topic, "Equipment and Methods for Maintaining Bituminous Surfaces and Bituminous Pavements", was presented by W. R. Farrington, M. Am. Soc. C. E. The topic was discussed by Messrs. Arthur H. Blanchard, Herbert Spencer, W. H. Fulweiler, and H. B. Pullar. J. A. Johnston, M. Am. Soc. C. E., took the chair, and the discussion was continued by Messrs. William de H. Washington, James H. Sturdevant, T. Hugh Boorman, and Philip P. Sharples.

Adjourned.

REGULAR MEETING

February 4th, 1914.—The meeting was called to order at 8.30 P. M.; Director T. Kennard Thomson in the chair; Chas. Warren Hunt, Secretary; and present, also, 186 members and 33 guests.

The minutes of the meetings of December 17th, 1913, and January 7th, 1914, were approved as printed in *Proceedings* for January, 1914.

A paper by Edward Flad, M. Am. Soc. C. E., entitled "Reinforced Concrete Reservoir and Coagulation Plant at St. Louis, Mo.", was presented by title, and discussed by Messrs. A. C. Janni and Edward Wegmann. The Secretary reported that he had received communications on the subject from Messrs. J. K. Finch and Alexander Potter. These were not presented on account of lack of time.

George W. Tillson, M. Am. Soc. C. E., addressed the meeting on "Some Observations on Pavements in Foreign Cities", illustrating his remarks with many lantern slides.

The Secretary announced the election of the following candidates on February 4th, 1914:

As Members

Henry David Alexander, Albany, N. Y.
Thomas Warren Allen, Washington, D. C.
Charles Joseph Bennett, Hartford, Conn.
Josiah Ackermann Briggs, Jr., New York City
George Condit Haydon, Kansas City, Mo.
Andrew Fullerton Macallum, Hamilton, Ont., Canada
Virgil George Marani, Cleveland, Ohio
James Bond Pope, San Francisco, Cal.
Hermann Frederick August Schussler, San Francisco, Cal.
Jacob Stephen Spiker, Vincennes, Ind.
John Williams Storrs, Concord, N. H.

As Associate Members

Otto William Boers, Chillicothe, Ill.
Charles Barto Brown, Orono, Me.
Clark Albert Bryan, Carlisle, Pa.
Edward Carter Chamberlin, Portland, Ore.
William Harrison Crawford, Philadelphia, Pa.
Allen Edrick Elliott, New York City
Harry Fallon Harris, Trenton, N. J.
Oliver Whitcomb Hartwell, Albany, N. Y.
Percy Francis Jones, Milpitas, Cal.
Raymond Brown Kittredge, Iowa City, Iowa
Frank B Kendall, Brantford, Ont., Canada
James Walter Martin, Georgetown, S. C.
Benjamin Baldwin Meriwether, Birmingham, Ala.
James Cecil Muir, New York City
George Washington Philips, Flushing, N. Y.
Walter Mitchell Pratt, Chicago, Ill.
William LeRoy Reynolds, Sheridan, Wyo.
Harrison Smith, Clanton, Ala.
Walter Lynes Smith, Ben Avon, Pa.
William George Boland Thompson, Balboa, Canal Zone, Panama
Thomas Leopold Tomlines, Watertown, N. Y.
Ernest Milton Trefethen, Albany, N. Y.
Robert Hoadley Whipple, Philadelphia, Pa.
John Wilkes, Nashville, Tenn.
Frederick Charles Young, Iowa City, Iowa

As Associate

Edward Hoffman Ailes, Corpus Christi, Tex.

As Juniors

Quincy Claude Ayres, Greenville, Miss.
Clarence Reed Carter, Houston, Tex.
William James Chouinard, Pocatello, Idaho
Curtis Fields Columbia, Palmerton, Pa.
Merton Arthur Darville, Brooklyn, N. Y.
Richard de Charms, Jr., Darlen, Ga.
Harry John Fairbanks, Troy, N. Y.
Allen Hoar, Long Beach, Cal.
Leon David Howland, LaGrande, Ore.
William James Henry Manning, Albany, N. Y.
Warren Shepard Matthews, New York City
Franklin Rufus Maxwell, Douglas, Ariz.
Joseph Holloway Morgan, New York City
David Leonard Neuman, New York City
Samuel Frank Newkirk, Stoneboro, Pa.
Robert Aldridge Nowlin, Eckman, W. Va.
Herbert Malcolm Pirnie, Beverly, Mass.
Walter Clifford Sadler, Mohler, Wash.
Morton Edwin Souther, Minneapolis, Minn.
Melvin Oliver Sylliaasen, Seattle, Wash.
Henry Ten Hagen, Nunda, N. Y.
Frank Martin Townsend, Eureka, Mont.
Theodore Ladd Welles, Jr., Wilmington, N. C.
Charles William Whitmore, Boston, Mass.
Clement Tehle Wiskocil, Madison, Wis.

The Secretary announced the transfer of the following candidates on February 4th, 1914:

From Associate Member to Member

Arthur Raymond Baylis, New York City
Webster Lance Benham, Oklahoma, Okla.
Horace Corey Booz, Philadelphia, Pa.
Arthur Maximillien Bouillon, Quebec, Que., Canada
Guy Whitmore Culgin, New York City
Walter Scott Gearhart, Manhattan, Kans.
Alvin Gravelle, Cleveland, Ohio
Louis Amedee Gueringer, Victoria, Tex.
William Albert Hansell, Atlanta, Ga.
Herbert Prescott Linnell, Manila, Philippine Islands
Franklyn Dana Nash, Clinton, La.
John Francis Sullivan, New York City

From Junior to Associate Member

Thomas Sherwood Bailey, Schenectady, N. Y.
Herbert Herluin Cantwell, Castorland, N. Y.
Omer Evert Malsbury, Culebra, Canal Zone, Panama
Garfield Christian Peterson, Glenbeulah, Wis.
Edwin Loring Sprague, Jr., Valhalla, N. Y.
Foster Towle, St. Ignatius, Mont.

The Secretary announced the following deaths:

Ricardo Manuel Arango, of Panama, Panama, elected Associate Member, September 2d, 1896; Member, February 6th, 1906; died January 24th, 1914.

John Willis Hays, of Petersburg, Va., elected Member, June 5th, 1901; died December 14th, 1913.

Peter Alexander Peterson, of Montreal, Que., Canada, elected Member, January 5th, 1876; date of death unknown.

Wallace Berkley Riegner, of Philadelphia, Pa., elected Member, September 7th, 1904; died January 19th, 1914.

Edgar Henry Mix, of San Francisco, Cal., elected Associate Member, April 2d, 1913; died January 7th, 1914.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

January 21st, 1914.—The Board met, as required by the Constitution, at the House of the Society during the Annual Meeting, January 21st, 1914, at 1 p. m.; President McDonald in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Edwards, Endicott, Fuller, Gerber, Haskell, Hedges, Hodge, Keefer, Leonard, Lewis, Loweth, Montfort, Ockerson, Rust, Smith, Swain, Thomson, Tuttle, Wallace, and Williams.

The President announced that the first business was the election of a Secretary.

Mr. Hunt retired.

Chas. Warren Hunt was nominated for Secretary and elected.

Mr. Hunt was recalled.

The following Standing Committees of the Board were appointed:

Finance Committee: Lincoln Bush, Henry W. Hodge, Leonard Metcalf, Emil Gerber, and George W. Fuller.

Publication Committee: James H. Edwards, William Cain, Arthur S. Tuttle, Henry R. Leonard, and Gardner S. Williams.

Library Committee: J. Waldo Smith, T. Kennard Thomson, E. C. Lewis, M. E. Cooley, and Chas. Warren Hunt.

The following resolution was adopted: That mileage be allowed to all members of the Board attending the meeting of the Board at the Annual Meeting, and that this action take effect as of January 1st, 1914.

Adjourned.

February 4th, 1914.—The Board met at 3 P. M.; Vice-President Smith in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Edwards, Endicott, Fuller, Gerber, Hodge, Keefer, Loweth, Thomson, and Tuttle.

The following Resolutions were presented from the General Conference Committee of the five National Engineering Societies:

"*Resolved:* That the General Conference Committee of National Engineering Societies approves the memorial presented to the President of the United States by the American Institute of Consulting Engineers in reference to the appointment of an Engineer member of the Interstate Commerce Commission, and recommends that each of the several National Societies take such action as it may see fit in the premises."

"*Resolved:* That in regard to the memorial presented by the American Institute of Consulting Engineers to the Governor of New York State, and to the Mayor-elect of New York City, urging the appointment of Engineers as members of the Public Service Commissions and as heads of City Departments, the General Conference Committee does not believe it to be the province of National Engineering Societies to take up questions relating to local appointments, and the Representatives of the various National Societies are hereby requested to so report to their respective Councils for such individual action as may be necessary in the premises."

On motion, duly seconded, the Report of the Committee was received and its recommendations adopted as the action of this Board.

Messrs. George W. Tillson, Nelson P. Lewis, and Charles J. Tilden, were appointed as additional members of the Special Committee on Bituminous Materials for Road Construction.

The time for the Annual Convention to be held at Baltimore, Md., was fixed as June 2d to 5th, inclusive. Committees of Arrangements were appointed.

The evening of Wednesday, March 11th, 1914, was set down for the discussion of the Progress Report of the Special Committee on the Valuation of Public Utilities.

Ballots for membership were canvassed, resulting in the election of 11 Members, 25 Associate Members, 1 Associate, 25 Juniors, and the transfer of 6 Juniors to the grade of Associate Member.

Twelve Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

REPORT IN FULL OF THE SIXTY-FIRST ANNUAL MEETING, JANUARY 21ST AND 22D, 1914

Wednesday, January 21st, 1914 (10 A. M.)—George F. Swain, President, in the chair; Charles Warren Hunt, Secretary; and present, also, about 600 members.

Meeting called to order.

THE PRESIDENT.—The meeting will please come to order. The first business matter to be attended to is the appointment of tellers to canvass the ballots for officers, and the President will appoint the following as tellers: Messrs. A. H. Van Cleve, J. H. Granbery, Asa E. Phillips, Frank G. Wolfe, Frederic S. Stow, and Merritt H. Smith.

Tellers Appointed.

THE SECRETARY.—Mr. President, the ballots are all ready on the third floor with the tally sheets and the full office force to help out. There are only about 1 700 ballots, and it is hoped that the tellers will be able to get through in about two hours. If they will go right upstairs they will find all the necessary help.

Report of the Board of Direction.

THE PRESIDENT.—The next in the order of business is the Annual Report of the Board of Direction, which will be presented by the Secretary.

The Secretary presented the Report of the Board of Direction.*

THE PRESIDENT.—You have heard the report of the Board of Direction. What is your pleasure?

It was duly moved and seconded that the report of the Board of Direction be received and filed.

THE PRESIDENT.—Gentlemen, you have heard the motion that the report of the Board of Direction be received and filed. Those in favor please say "aye"; contrary-minded, "no". Carried.

The next business is the Report of the Secretary.

Report of the Secretary.

THE SECRETARY.—Mr. President, this is the report of the Secretary to the Board of Direction.

The Secretary presented his Report† of receipts and disbursements for the year.

Part of that report, Mr. President, is the general balance sheet,‡ which shows the assets and liabilities of the Society, which perhaps it is not necessary to read, but which shows that the surplus, including the Reserve Fund, which would represent what the Society could sell out for, is $644 829.57.

THE PRESIDENT.—This is a report of the Secretary to the Board of Direction, and requires no action of the Society, unless the Society desires to take some.

Report of the Treasurer.

The next business is the Report of the Treasurer, Mr. Wallace.

The Treasurer read his Report.§

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* See *Proceedings*, Am. Soc. C. E., Vol. XL, p. 11 (January, 1914).
† See *Proceedings*, Am. Soc. C. E., Vol. XL, p. 20 (January, 1914).
‡ See *Proceedings*, Am. Soc. C. E., Vol. XL, p. 19 (January, 1914).
§ See *Proceedings*, Am. Soc. C. E., Vol. XL, p. 22 (January, 1914).

The President.—You have heard the report of the Treasurer. What is your pleasure?

It was moved and seconded that it be accepted, and filed.

The President.—It is moved and seconded that the report be accepted and filed. Those in favor of the motion, please say "aye"; contrary-minded, "no". Carried.

The next business is the report of the Committee appointed by the Annual Convention last summer to report to this Annual Meeting on certain amendments to the Constitution. This report is to be presented by the Chairman of the Committee, Mr. Endicott.

Report of the Committee on Amendments to the Constitution.

Mordecai T. Endicott, Past-President, Am. Soc. C. E.—Mr. President, the report of the Committee as printed I presume has been distributed, so that all members have a copy. By referring to the last page, 10, you will notice that the report is submitted by four of the five members of the Committee. The report is not signed by Mr. C. F. Loweth. All the other members unite in the report.

One of the principal purposes of referring this matter to the Committee, as we understand it, was to harmonize the several amendments before the Business Meeting of the Annual Convention last summer, particularly those relating to the redistricting of the territory occupied by members of the Society; and, as stated in this report, some of you will recall that in the consideration of the first amendment, called the Keefer amendment, upon the first and second pages of this report, the Business Meeting at the Annual Convention expressed its approval of the first section of that amendment, that is, taking it up section by section, and at that point the question arose whether it should consider these amendments under the Keefer proposed amendments, and those proposed by O. E. Hovey and others, together, and during that discussion the matter was referred to the Committee, and we have endeavored to thrash those into line; and inasmuch as the Business Meeting had expressed its approval in substance of what we call the Keefer amendments, there being no objection to it, your Committee recommends that no amendments to this amendment be made, which apparently meets with the sanction of the Business Meeting referring the several amendments to us, and recommends that it be sent out to letter-ballot without change.

If the meeting is to consider these separately, at this stage, it is proper to consider whether it will endorse the recommendation of the Committee and send this out to letter-ballot without change. As you see, it refers to redistricting, giving us fourteen districts instead of seven, the members of the Nominating Committee remaining the same.

The Secretary.—Thirteen.

Discussion on Amendments (continued).

Mr. Endicott.—Yes, thirteen. I beg pardon, No. 1 is a double-header. I move that the amendments proposed by Keefer and others shown on pages 1 and 2 of this report be sent out to letter-ballot without change.

F. P. Stearns, Past-President, Am. Soc. C. E.—I second the motion.

The President.—It is moved that these amendments proposed by Keefer and others be sent out to letter-ballot without change. Those in favor of the motion say "aye"; contrary-minded, "no". Carried.

Mr. Endicott.—Then there were amendments proposed by Hovey and others, which were also considered by the Business Meeting to which I referred, in Ottawa. Those go into the question of whether there should be a change in the status of the Secretary with respect to the Board of Direction, and also with respect to the matter of redistricting, as to the method of the selection or election of the Nominating Committee, and it was those amendments which threw the meeting into some confusion.

The proposed Hovey amendments occur on pages 3, 4, 5, and 6. They are printed there exactly as they were laid before the Business Meeting of the Annual Convention last summer, and, beginning at page 6, and on, this Committee discusses them somewhat and states certain conditions which you have probably read.

First, the Committee recommends that these be not voted upon collectively, because the subjects treated are so very much at variance, and particularly that relating to the Secretary of the Society, which I believe should be voted on independently of all other proposed amendments. Those are Articles V and VI, referring particularly to his status and the manner of his election. Under Article V there were two amendments proposed, Section 1 and Section 2, and the Committee recommends that the amendments proposed by Hovey and others to Article V, Section 1, and Article V, Section 2, first paragraph and second paragraph, stand unchanged.

The President.—And be sent out to letter-ballot?

Mr. Endicott.—And be sent out to letter-ballot in that form. Now, we recommend that the amendments to Article V, Sections 1 and 2, be voted upon separately. Article V, both Section 1 and Section 2, should, in our opinion, go together. I move that the amendments, Article V, Sections 1 and 2, go out to letter-ballot unchanged as recommended by the Committee.

Motion duly seconded.

The President.—It is moved and seconded that, as recommended by the Committee, the proposed amendments to Article V, Sections 1 and 2, go out to letter-ballot unchanged and be voted upon together.

Those in favor of the motion say "aye"; contrary-minded, "no".

J. F. Wallace, Treasurer, Am. Soc. C. E.—Am I privileged to ask a question?

The President.—Yes.

Mr. Wallace.—I am rather confused on this. I want to know if this motion refers to the status of the Secretary or does the motion refer to the first two amendments in regard to the nomination and election of officers? Are these amendments to Articles V and VI?

The President.—Article V.

Mr. Endicott.—The amendments to Article V only.

Mr. Wallace.—That is what I am trying to find out, what we are voting on.

Mr. Endicott.—Article V only.

Mr. Wallace.—I move an amendment to that motion that these amendments be referred back to the Committee with instructions that it shall report sixty days prior to the next Annual Convention.

A Member.—I second the motion, that is, the amendment to the original motion I second.

The President.—It has been moved and seconded that this be referred back to the Committee with instructions to report not less than sixty days prior to the next Annual Convention. Perhaps Mr. Wallace will explain the reasons for that motion.

Mr. Wallace.—Well, these amendments proposed are radical changes in the methods of handling the business of this organization. No organization as large as ours, and as successful as ours has been, can be handled without a continuity of management. The Society gets that continuity of management through a continuing Board of Direction, one-third of which is elected each year. We have got to-day a surplus of $650 000, practically, and we are prosperous and growing; and as I have resided a short time in Missouri, I would like to be shown the necessity of any change in our policy.

This is an age of restlessness; it is an age of change; it is an era of experiments. In our political life we see to-day the Constitution of the United States, which is the flower of the best Anglo-Saxon thought for a thousand years, subjected to criticism. The tendency is to criticize, to tear apart.

I have served on the Board of Direction of your Society ten years, two as Vice-President, one as President, five as Past-President, an additional year as Past-President owing to the death of one of my predecessors, and during this last year as Treasurer, and I feel that your present methods of continuing the management of the Society, holding your Board of Direction responsible for results, have not resulted in anything that should be criticized or changed. I feel that every officer of this Society should be under that Board of Direction, should hold his office subject to that Board of Direction, and not by the popular vote of the Society.

Discussion on Amendments (continued).

If we could all get together on stated occasions, frequent occasions, and were not so widely scattered over the country, it might be entirely different; but if you gentlemen want to see the prosperity of this Society continued as in the past, you will make no present change in the condition of this organization. My desire to see this thing referred back is this: First, this report on which we are called to act was put out under date of January 16th. I think that before a matter of this importance should be sent out to letter-ballot it should have the ample, full, and free discussion of every member of this Society; and that is my reason not only for moving this amendment, but for urging it. (Applause.)

C. S. Churchill, M. Am. Soc. C. E.—I am in full accord with the proposed action of Mr. Wallace. The organization of this Society cannot be, should not be, destroyed in the manner proposed, and I think that this vote should be taken if for nothing more than to show the sense of this Annual Meeting that we are opposed to the change.

F. W. Hodgdon, M. Am. Soc. C. E.—Mr. President, I would like to ask for information. Has this report been accepted by the meeting?

The President.—The report is now before the meeting for such action as the meeting may take. It has been presented.

Mr. Endicott.—It is being presented paragraph by paragraph, with the recommendations of the Committee, for action.

A Member.—I think before we take any action we should accept the report.

The President.—Do you make that motion?

A Member.—I am heartily in favor of Mr. Wallace's suggestion that this whole thing be put over until the membership of the Society has had a full opportunity to know what it means. I do not think that, before the report is accepted by the meeting, we can take any action.

A Member.—I understand that to be Mr. Wallace's motion, that the report be accepted and referred back.

Mr. Wallace.—My motion was an amendment on the motion to refer it back.

A Member.—Your preliminary statement that the report be accepted and referred back——

Mr. Wallace.—If you will permit me, I think the most direct way to handle this would be to withdraw the motion and the amendment and then make a motion to accept the report and discharge the Committee, and then take any action that the Society desires to take on the suggestions.

A Member.—The report cannot be amended or acted upon until it is received; at least, the point I make is not necessarily an acceptance of the report in its terms, because I am heartily in favor of time to understand this report. I do not understand it. Mr. Wallace, of course, knows more about it than I do, because he is on the

Board, but as an outsider, coming from a distance, the report is mysterious and uncertain to me, and therefore I am heartily in favor of referring it back, but it should be received first, before anything can be done.

Mr. Wallace.—If the gentleman who made the first motion to which mine was an amendment, will withdraw that motion, I will withdraw the amendment. I will then move that the report be received and the Committee discharged.

D. FitzGerald, Past-President, Am. Soc. C. E.—Mr. President, as there is evidently a difference of opinion in regard to this matter, in the interest of fair play, I move, sir, that this report be accepted. That is the proper way to keep the matter. Then it can be discussed. You can do what you choose with it. I move that it be accepted.

Motion duly seconded.

The President.—It is moved and seconded that the report be received——

A Member.—I rise to a point of order. We have already passed upon one part of it, and another part has been properly presented with the recommendation. I think it is a perfectly proper proceeding. We may decide to receive part of it, and reject part of it.

The President.—The chair will sustain the point of order.

J. A. Ockerson, Past-President, Am. Soc. C. E.—It seems to me that an explanation is in order. Mr. Wallace says it is a new subject, which is presented under date of January 16th, 1914. It was presented at the Annual Convention by five members and properly signed as a proposed amendment to the Constitution. The Committee's function was to bring together two sets of amendments, so that they would be consistent with themselves and with the other portions of the Constitution, and it seems to me that the five members have a right to demand that it should go out to the Society for ballot. It is not a question of sustaining the Committee at all. It is a matter that has been properly presented, and the Society should have the right to act upon it.

Mr. Stearns.—Mr. President, I would like to say a few words more in explanation. As Mr. Ockerson has said, this is a matter that has been before the Society for some time, but the point to which I wish to call attention is this, that practically the only amendment—and this is the only change from the former language—is to put in the words "except the Secretary". Now, the Committee considered that without any regard to their own opinions, and looked at the Constitution which said that they could only amend it in a way pertinent to the proposed amendment. The amendment that they would have made, if any, would have been to strike out the words "except the Secretary". That would have left the Constitution as it

Discussion on Amendments (continued).

is. It evidently is not pertinent to make such an amendment as that. This matter has to go out, under the provision of the Constitution, to the members of the Society. If this motion that is made passes, it will be put off for six months. That is all it will amount to, as I understand the matter. You cannot amend a proposed amendment like that in a manner that is pertinent to the original amendment.

Mr. Wallace.—Mr. President, I was not at the Annual Convention, and I presume that is my fault; but I do not see how the interests of the Society can suffer in any way whatever by the postponement of the consideration of this question and by carrying out the suggestions embodied in my amendment. If it were something that required immediate action, if the advantages of it were so preponderant, it would be entirely another matter; but here we are purporting to take a subject, to overturn practically our entire policy, or what amounts to the same thing, in the management of this organization, and I do not see why we cannot all have proper time to do it.

There is one clause that seems to me—it may be my own fault, but I have not seen this report, which is dated January 16th—and it does seem to me that we should have plenty of time, and that no interests can be prejudiced in any way by taking six months longer to consider this proposition. All I ask is that it be referred back to the Committee, and that it be formulated, and that we have sixty days in which to consider it, and to consider it at the Annual Convention, where we always have a much larger attendance than we do at our Annual Meetings here.

Mr. Stearns.—The remarks which I made were only aimed at the futility of sending that particular thing back to the Committee. I do not intend to express any opinion in regard to the ballot. If the Society wishes, it can send it back to the Committee. In that particular respect, as a matter that in my judgment cannot be amended, it seems useless to do so.

(Cries of "Question".)

Allen Hazen, M. Am. Soc. C. E.—Mr. President, as a member of this Committee, and with the prospect of having to face this matter again, I think the function of the Committee in considering it should be determined more definitely. The Constitution provides that an amendment to the Constitution may not be amended by the meeting in a way contrary to the effect of the amendment. The Constitution provides as an alternative that these matters may be referred to a Committee. I do not find anything in the Constitution which limits the scope of the consideration which the Committee may give to it. However, the Committee of which I was a member decided, after due discussion, that the Committee was limited by the same principles that limit the action of the Annual Meeting. Under that decision our only function was a literary one, to shape the language of it so

that it would be a proper part of the Constitution if it was adopted. The Committee proceeded on that basis, I think. The amendments were harmonious and put in proper shape if adopted.

I do not understand that this action of the Committee under this rule expresses any opinion of the Committee whatever, as to the advisability of the amendments. Aside from any final action, it is my judgment that a majority of the Committee was strongly against some of the amendments included in this report, and I think that fact should be duly recorded, and if the matter is referred to the Committee again, I would like to have the fact made very clear, whether it is simply a matter of language that is referred, or whether the Society desires the judgment of the Committee on the advisability of the proposed change. (Applause.)

Mr. Endicott.—Mr. President, I am unable to see the force or value of the proposal of Past-President Wallace, because, if this meeting sends this out to letter-ballot, it is sent out by the Board of Direction to every member of this Society, every Corporate Member, everybody who can vote upon it. He will have until next October to consider that matter. The Constitution provides that an amendment shall be voted on not later than October, I believe, following.

The Secretary.—I beg pardon. I think the amendments could be voted on in March of this year, under the Constitution.

Mr. Endicott.—That would give them practically three months. The proper way to get it before the Society is to send it out in the form of the proposed amendment, as this meeting may amend it under its authority, and leave it in the hands of the members of the Society for two or three months, and then there is full time to consider it. I do not see any advantage at all in postponing the consideration of this matter for six months. We would be then just where we are to-day.

C. F. Loweth, M. Am. Soc. C. E.—Mr. President, as a member of this Committee, I did not sign this report, because it seemed to me that the Committee had not gone into it with the thoroughness that should have been done. It seems to me that it was pertinent for the Committee to come before this meeting with a recommendation, and it has not done so. There are a number of these amendments that are not in such shape as they should be. They are contradictory. They do not line up with the rest of the Constitution, and it would be a very easy matter to make those amendments, to make those changes, so that if those amendments were carried they would properly fit into the Constitution. I do not believe that that has been done, and then I think it was proper and pertinent to the Committee to have made some statement in its report as to the choice of those two amendments, the one that has been offered by Mr. Keefer and the one that was offered by Mr. Hovey.

Discussion on Amendments (continued).

Now, they cover the same ground, and why are they put up to the membership to vote on? It is likely to make confusion. It seems to me that the Committee has not gone into its work and covered it thoroughly, as the meeting had a right to expect it to do.

T. K. Thomson, M. Am. Soc. C. E.—Mr. President, as a member of the Board of Direction, I think I might be asked if I know anything about this proposed change. I attended most of the meetings, and I do not think there was any meeting where I could have had any chance to explain it. I got into the room too late to read it, and all I know is what I have heard.

I think any important amendment should be thoroughly discussed by the Board of Direction before being submitted to the Society. That is one reason I so strongly endorse what ex-President Wallace has just said. I might say that while I have only had the honor of being on the Board for two years, in the last twenty-one years I think I have attended more meetings than any other member of the Society, and I do not think anybody thinks we have reached perfection yet, but there has been a steady improvement right along, and it has been very marked in the last few years; and I sincerely hope you will carry out Past-President Wallace's suggestion in taking plenty of time to consider such an important matter.

(Cries of "Question".)

S. Whinery, M. Am. Soc. C. E.—Mr. President, I wish to question the very narrow definition that has been given by some gentlemen of the word "pertinent". I do not think it was ever intended, or is now intended, or should be the rule, that the word "pertinent" should mean simply verbal constructions. Having been a member long ago of the Committee that revised the Constitution, and I think re-wrote this very section, I think I may say that certainly it was the intention of the Committee that the word "pertinent" should be defined to mean pertinent to the subject, not merely to the narrow question of the verbal construction, or not confined merely to the section of the proposed amendment. In other words, I believe that this meeting and this Society has a perfect right to deal with this matter with reference to the whole general subject, anything pertaining to this general question that has been presented to the Society; and I think in that way it is proper for this meeting to refer the question back, or the proposed amendments that relate to the general subject here treated of.

The President.—It may conduce to clearness and save some time of the meeting, if I read the provisions of the Constitution with reference to this matter. The Constitution provides that:

"Amendments presented to the Secretary not less than sixty days previous to the date of the Annual Convention shall be sent by letter to the several Corporate Members of the Society, at least twenty-five days previous to the Annual Convention."

That was done last summer. "Said amendments shall be in order for discussion at the Business Meeting during such Annual Convention," Those were discussed at the Annual Convention. "and, if so amended, shall be voted upon by letter-ballot in form as amended by said Business Meeting; if not so amended * * * The vote to be counted at the first regular meeting in October."

The next paragraph refers to the fact that amendments may be brought up a short time before the Annual Convention or a short time before the Annual Meeting, and this was brought up a short time before the Annual Convention last summer——

"It may refer the amendment to a Committee for further consideration, which Committee shall report at the next general meeting, whereupon the amendment shall be voted upon as hereinbefore provided."

Now, that is just what was done at the Annual Convention. This matter was discussed. It was referred to this Committee for further consideration. This Committee has referred this report to you, and the Constitution says "whereupon the amendment shall be voted upon." The chair is of the opinion that, according to that, the recommendations of this Committee have got to go to the membership to be voted upon, and cannot be postponed. The Constitution does not say that it may be amended at this meeting even. It says that after discussion at the meeting at which amendments are properly presented, the Society, by a majority vote, may refer them to a Committee, which was done, and that Committee shall report at the next general meeting, which is this meeting; whereupon the amendments shall be voted upon as hereinbefore provided.

It appears to the chair, as I said before, that these have got to be voted upon as recommended by the Committee. The Society may vote them all down, but it appears to me—I am speaking of the way I interpret the Constitution—it appears to me they have got to go to a vote now.

W. L. Saunders, M. Am. Soc. C. E.—Mr. President, may I ask if this report was presented to the Board of Direction sixty days before this meeting?

The President.—It was not. It was presented sixty days before the last Convention.

Mr. Saunders.—I know that, but this report, as I see it, does not follow fully and strictly the previous report. This report makes certain suggestions which were amendments to those amendments. As I read this report, they do not agree in everything with the previous amendments. If that is the case, and I think it is so written here, it seems to me that having changed the previous Committee's report, that this report, after it shall have been presented sixty days before this meeting——

Discussion on Amendments (continued).

THE PRESIDENT.—There was no previous Committee report. The amendments were presented at the last convention. They were inconsistent with each other, and the convention got into a snarl because they were inconsistent. They were referred to a Committee. The Constitution does not provide that that Committee shall report to the Board of Direction. It says distinctly "shall report at the next general meeting," which it has done to-day, whereupon, the Constitution says, the amendments shall be voted upon as hereinbefore provided.

MR. SAUNDERS.—My point is that this report itself should have been presented to the Board of Direction sixty days before this meeting.

THE PRESIDENT.—There is nothing in the Constitution that provides that that should be done.

H. W. HODGE, M. AM. SOC. C. E.—I rise to the same point of order that the President has very plainly stated. I cannot see, under this Constitution, that this meeting can do anything with these amendments, and before I leave that point I want to say that this has been before the Board of Direction. The Board of Direction has considered it. Four out of five of the Committee have recommended it. What more can the Board of Direction do; but I submit that this motion to postpone is out of order, and I would ask the chair to so rule, and have the ballots sent out to vote.

THE PRESIDENT.—The chairman has stated very much in detail the way the matter appears to him, according to the Constitution, and I therefore rule that Mr. Wallace's motion is out of order.

MR. CHURCHILL.—Mr. President, I move that it is the sense of this meeting, this Annual Meeting, that the proposed motion or amendments to Article V, Sections 1 and 2, containing the three words "except the Secretary" are not proper and not for the interests of this Society.

A MEMBER.—I rise to a point of order there. As I understand it, the question now before the House is sending this out to a letter-ballot, and while that is a proper subject to take up later, I move it is out of order now.

THE PRESIDENT.—The chair was about to make the same ruling. The motion before the Annual Meeting is that the amendments to Article V be sent out to letter-ballot unchanged. If Mr. Churchill, later, wishes to make the motion that it is the sense of this meeting that the amendments are undesirable, that is perfectly proper, as an expression of the sense of this meeting.

F. S. CURTIS, M. AM. SOC. C. E.—I want to take exception to the President's ruling that Mr. Wallace's motion is out of order. It is true that these should be sent out by letter-ballot by this meeting to determine—and this motion was an amendment to the original motion to refer it back to the Committee. I cannot understand why that is out of order, that the Committee should reconsider and consider

further this matter, which, as I understand, the Committee is not unanimous upon in its report, and it is thrown upon us, a good many of us, who know very little about it, and we should go slow and not too fast, and I cannot quite understand why Mr. Wallace's motion is out of order, and I do not understand why you, Mr. President, should so decide.

John Bogart, M. Am. Soc. C. E.—Mr. President, may I ask a question? Has the report of this Committee been read to this meeting?

The President.—It is being presented paragraph by paragraph, or recommendation by recommendation, by the Chairman of the Committee.

Mr. Bogart.—That I understood for the first two pages here. It was then moved, as I understand it, that that part be sent out to letter-ballot. Now, we have jumped into something else, and certainly that part has not been read, and a number of us have not these papers. I just this moment got hold of one.

The President.—The Chairman of the Committee will be glad to read them if the Society desires.

Mr. Bogart.—I believe we ought to have the whole thing read.

Mr. Endicott.—Mr. President, when I presented them first, I referred to this first part being in the hands of all, and supposed they had read it and therefore thought it was unnecessary to take it up after the meeting had read it. The amendments referred to as proposed by Hovey and others are as follows:

"Article V, Section 1.—Strike out the first sentence and substitute the following:"

You will find the original Article on page 10 of the Constitution.

Mr. Bogart.—Will you explain why they are proposed?

Mr. Endicott.—I have already stated that the object of the amendments is to change the status of the Secretary, so that if this amendment passed hereafter he would not be a member of the Board of Direction. That is the only change proposed in Article V.

"The officers of the Society shall be a President, four Vice-Presidents, eighteen Directors, a Secretary, and a Treasurer. These officers, except the Secretary,"—that is where the amendment comes in—"except the Secretary". Those words are not in the original Constitution; those three words constitute the amendment proposed by Hovey and others. "These officers, except the Secretary, together with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction in which the government of the Society shall be vested, and who shall be the Trustees as provided for by the laws under which the Society is organized."

And in Article V, Section 2, First Paragraph, there is a slight amendment, which becomes necessary, if the amendment to Section 1 is passed. It says:

Discussion on Amendments (continued).

"The terms of office of the President and Treasurer shall be one year; of the Vice-Presidents, two years; and of the Directors, three years."

The Constitution says that the terms of the President, Secretary, and the Treasurer shall be one year. There are amendments which follow, which say that the Secretary shall serve until his successor is elected, instead of for the term of one year. So that his term of office would not necessarily be one year. That is the whole of the amendment to Article V, and the Committee has recommended first the amendments to Article V, which refers only to the membership of the Secretary in the Board of Direction.

THE SECRETARY.—There is one other change in Article VI, Section 4.

THE PRESIDENT.—The Secretary has called my attention to the fact that there is also an amendment proposed in the second paragraph of Article V, Section 2, as follows: "In first line insert after the word 'officer' the words 'elected by the Society' so that the paragraph shall read:

"The term of each officer elected by the Society shall begin at the close of the Annual Meeting at which such officer is elected, and shall continue for the period above named or until a successor is duly elected."

A. S. TUTTLE, M. AM. SOC. C. E.—Mr. President, it seems to me, speaking as one of the members, we have had this before us for quite some time. I think each individual member has given it the consideration that he ought to have given it. He has had ample time at any rate to do so, and in view of that fact, I move the question.

A MEMBER.—I rise to a question of privilege. I believe a motion to lay on the table is always in order, and I would therefore move that the motion be laid on the table.

THE PRESIDENT.—The chair will rule that motion out of order. The question before the meeting is that the proposed amendments to Article V, Sections 1 and 2, be sent out to letter-ballot unchanged.

(Cries of "Question.")

A MEMBER.—I think we are entitled to know whether the Committee is reporting as a whole on the English that is used in phrasing these amendments, or on the desirability of the matter——

THE PRESIDENT.—It is not for the Committee or the meeting to express itself as to the desirability, except informally. This meeting may later express its opinion as to whether the amendments ought to pass.

GARDNER S. WILLIAMS, M. AM. SOC. C. E.—Mr. President, under your ruling, is it necessary for this meeting to vote on the question at all? When this report is received, does not this automatically go

out to letter-ballot, and is not that the end of it? And if this meeting wants to express an opinion as to whether the amendments should pass or not it may do so. I would ask a ruling by the chair as to whether any action is necessary at this meeting. I think we are wasting time. These amendments have been properly proposed and properly acted upon by the Board of Direction, and there is nothing left under the Constitution but to send them out to letter-ballot. I would ask the chair to rule that there is no motion necessary, and that these amendments do go out to letter-ballot under the Constitution.

The President.—That is the opinion of the chairman.

A Member.—Inasmuch as there is some question on the chairman's rulings, does that ruling stand?

The President.—The chair, then, in order to save time, will rule that the amendments proposed to the Constitution, in the form recommended by the Committee, which has just reported, now go out to letter-ballot of the Society.

Mr. FitzGerald.—Mr. President, may I beg you to consider, it is very important that we should get to a full understanding of this matter. Under your former ruling, and under the Constitution, we are obliged to vote on this matter, and we cannot send it out automatically. It is impossible. This matter is not before the meeting until it is received, and then we have got to vote on it.

A Member.—I rise to a point of order. There is no motion before this meeting except the ruling of the chair; no other discussion is in order.

J. Waldo Smith, M. Am. Soc. C. E.—I would like a ruling on one question. This Committee was appointed at Ottawa to report at the next Annual Meeting of the Society. Does that mean that this meeting has got to accept that report, and has nothing to say about it? If that is so, why does that Committee report to this Annual Meeting? It seems to me that it was referred to this Annual Meeting in order that it might be discussed, accepted, or rejected, and the chairman—the chair—does not so understand the Constitution. As the chair understands the Constitution, any amendment to the Constitution may be proposed, and if it is proposed, if it is a legitimate amendment in wording or sense, if it is proposed by five members, it has got to go to letter-ballot and nobody can prevent it from going to letter-ballot. Now, these amendments were proposed at the last Annual Convention, but it so happened that they were inconsistent with each other. The several amendments then proposed were inconsistent with each other, and manifestly they could not very well go to letter-ballot without being made consistent, and they were referred to this Committee to be made consistent, so that amendments might go to the membership which would be intelligible and consistent. This Committee has now made them consistent, and reports to this

Discussion on Amendments (continued).

meeting, and the Constitution says that the Committee shall report at the next Meeting, whereupon the amendments shall be voted upon.

A MEMBER.—Then let us vote upon them.

THE PRESIDENT.—It does not say voted upon by the meeting, voted upon as hereinbefore provided, by the Society by letter-ballot, not by the meeting.

H. B. SEAMAN, M. AM. SOC. C. E.—Mr. President, I feel that all discussion of this matter has been very unfortunate and will be misunderstood. It will be construed as an effort to throttle the voice of the membership. The membership controls this Society, not any body of men, nor any individual. This vote must go to the membership, and our discussion here can only be construed as an effort to throttle that vote. If that amendment is bad, it will be voted down, and the membership have sense enough to do it; and if it is good, it will go through, and the membership have sense enough to do it; and it should go to the membership and not be decided here.

THE PRESIDENT.—The ruling of the chair, that the amendments to the Constitution as reported by the Committee shall go now to letter-ballot as provided by the Constitution is challenged, and you wish to have a vote on it. It will stand as the decision of the chair.

MR. HAZEN.—Mr. President, I think it ought to be understood very clearly, beyond any possible doubt, that the Committee recommends that they go to ballot. The Committee does not recommend that they pass. It does not recommend the amendments; and that cannot be too emphatically stated.

THE PRESIDENT.—Nobody has recommended the amendments. The Committee has not recommended that they pass or that they do not pass. The Committee has simply reported them to this meeting.

MR. CHURCHILL.—I move that it is the sense of this meeting that the suggested change to Article V, Section 1, is undesirable.

Motion duly seconded.

THE PRESIDENT.—Before that is put I would simply state that I understand that the ruling of the chair not having been challenged, these amendments now go to letter-ballot in the form now recommended by the Committee.

MR. CURTIS.—I move an appeal from the President's decision. I understand that the President of this Society says that we have nothing whatever to say about it except by letter-ballot. It is brought up before this meeting for consideration. Now, why? Why was it brought up? Why say anything about it at all? Why did not the President go ahead and order it sent out, and not bring it before this meeting. It is brought before us. I think we have a right to say something about it, as to whether it shall go out, or whether it shall not go out, and I move—as I have already said, I do not want to go

against our President, because I think very much of him—I move an appeal from his decision.

Motion duly seconded.

The President.—That is a perfectly proper motion for Mr. Curtis to make.

D. C. Jackson, M. Am. Soc. C. E.—This is not a parlimentary question; it is a constitutional question; and the vote of this meeting has nothing to do with a constitutional question. The literal statement of the Constitution as to that is that this Committee, which is now reporting, shall report to this meeting, and thereupon the amendments proposed shall go to letter-ballot. That ends it. This meeting can say nothing more about it.

The President.—The chair believes that Professor Jackson is right, and I am very much perplexed as to what the situation would be if the decision should be overruled, because, in my opinion, we would then be proceeding in an unconstitutional manner. (Laughter.) Shall the decision of the chair be sustained? Those in favor please hold up their hands. Those against. The decision of the chair is sustained. The amendments themselves will go to letter-ballot in the form recommended, all of them in the form recommended by the Committee.

The Secretary.—Exactly in the form?

The President.—In the form recommended by the Committee.

Hunter McDonald, M. Am. Soc. C. E.—Pardon me for calling attention to the omission of one word in this report, which if it should go out in that way, would cause a great deal of misunderstanding. About the middle of page 9 it appears that the——

Mr. Endicott.—We are not considering the portion you mention.

Mr. McDonald.—I would like to have the phraseology changed by the Committee.

The President.—You cannot change it.

Mr. Wallace.—The President has decided that this report shall go to letter-ballot. Therefore it is improper for us to discuss it in any way whatever. We will never get anywhere. We are simply wasting our time.

Mr. Davis.—I cannot see how it is out of order for this meeting to recommend something to the membership.

The President.—Not at all. The motion before the Society is that of Mr. Churchill, whose motion is that in the opinion of this meeting the recommendations—I am not quite clear as to whether he means to cover Article V. Will you please state your motion.

Mr. Churchill.—That the suggested change to Article V, Sections 1 and 2, are undesirable.

Motion seconded.

The President.—It is moved and seconded that, in the opinion

Discussion on Amendments (continued).

of this meeting, the amendments to Article V, Sections 1 and 2, are undesirable. Is there any discussion on that? Those in favor of the motion say "aye"; contrary-minded, "no". The chair is in doubt.

A MEMBER.—Will the chair please explain what the motion is.

THE PRESIDENT.—The changes in the Constitution contemplated by these amendments——

A MEMBER.—All of them?

THE PRESIDENT.—Article V, Sections 1 and 2. The motion, which was not conclusively settled by the *viva voce* vote, is this, that in the opinion of this meeting it is not desirable that the suggested amendments to Article V, Sections 1 and 2, should pass. Is that correct, Mr. Churchill?

MR. CHURCHILL.—Yes, sir.

THE PRESIDENT.—We were about to take a vote on that. Is there any one who does not understand that? You are not in order, later perhaps. Is there any misunderstanding on that?

A MEMBER.—Have I not any right to speak on that at all?

THE PRESIDENT.—No; the discussion is closed. Those in favor of that motion, that the amendments to this Article are undesirable, will please raise their hands. Those opposed.

THE SECRETARY.—90 to 122.

THE PRESIDENT.—It is not a vote. The motion is lost. The next business is the appointment of the Nominating Committee; Report of the Secretary regarding the Final Suggestions which have been received from the Seven Geographical Districts.

MR. J. WALDO SMITH.—There are some other articles.

THE PRESIDENT.—All those articles go to ballot.

MR. J. WALDO SMITH.—Then I would like to make a motion, if I may. At the Ottawa Convention there was a protest presented, signed by some 800 members of this Society, scattered throughout the length and breadth of the country, protesting against the amendments, the Hovey amendments affecting the standing of the Secretary, and also the manner of electing the Nominating Committee. In order that the general membership of this Society may have the benefit of the opinion of those people as expressed in their protest, I move that, when these amendments are sent out to letter-ballot, a copy of this protest with the signers accompany the letter-ballot.

THE PRESIDENT.—Was not that protest printed in the Proceedings of the Ottawa Convention? It has already been before the Society, Mr. Smith.

MR. J. WALDO SMITH.—In order that that may be particularly called to their attention, I make that motion.

Motion duly seconded.

MR. HODGE.—I strongly object to this meeting—I am not talking about the value of these amendments. Some of them may be desirable,

and some may not, but I do say that the Constitution does give the membership the right to an unbiased vote. The membership are entitled to an unbiased vote, and to send out the views of any side to this controversy I think is a mistake. Our membership are brilliant men, and I do not think that either side of this small meeting—600 members out of 6 000—should try to send out their views as to how the membership should vote. Let them vote their own way.

MR. ENDICOTT.—I hope that the proposition will not prevail. I think it would be a most dangerous precedent for this Society to set, that it could send out *ex parte* statements to influence the vote of the membership on questions of this kind.

THE PRESIDENT.—It has been duly moved by Mr. Smith, and seconded, that a statement opposing the adoption of certain amendments be sent out with the amendments. Those in favor of the motion will please say "aye"; contrary-minded "no". It is not a vote.

The next is suggestions regarding the Nominating Committee.

MR. SEAMAN.—Mr. President, I am told that we voted on a question as to the opinion of the members present that these amendments should not prevail, and many of us did not know whether we were voting for it or against it. I think the motion should be put again in the form that it is the sense of this meeting that these amendments should prevail, and let that go out.

THE PRESIDENT.—The chair rules that out of order. The question was clearly put; the question was asked if every one understood what he was voting on, and the vote was lost.

The Secretary will present the Report of the Nominating Committee. Shall we take this up by districts?

Nominating Committee.

THE SECRETARY.—I beg leave to report a count of the final suggestions received from members in the several districts expressing their choice for the member of the Nominating Committee to be appointed from each, as follows:

Total suggestions received from all districts, 1 946.

District No. 1: Total number of suggestions received, 407, as follows:

Lewis D. Rights	229
Ernest P. Goodrich	101
Charles J. Parker	74
Walter F. Whittemore	1
Ralph H. Chambers	1
William J. Wilgus	1

A MEMBER.—I move that Mr. L. D. Rights be named as a member of the Nominating Committee from District No. 1.

Motion seconded.

Nominating Committee (continued).

THE PRESIDENT.—It is moved and seconded that Mr. L. D. Rights be a member of the Nominating Committee from District No. 1. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—District No. 2: Total number of suggestions received, 230, as follows:

Joseph R. Worcester	86
Charles M. Spofford	71
Richard S. Lea	58
John C. Moses	14
Joseph H. O'Brien	1

A MEMBER.—I move that Mr. J. R. Worcester be named as a member of the Nominating Committee from District No. 2.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. J. R. Worcester be a member of the Nominating Committee from District No. 2. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—District No. 3: Total number of suggestions received, 245, as follows:

Frederick E. Turneaure	103
Frank R. Lanagan	100
William H. Yates	40
William S. Bacot	1
Henry N. Ogden	1

MR. WILLIAMS.—I move that Mr. Frederick E. Turneaure be selected as a member of the Nominating Committee from District No. 3.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Frederick E. Turneaure be a member of the Nominating Committee from District No. 3. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—District No. 4: Total number of suggestions received, 308, as follows:

Thomas Earle	136
Richard L. Humphrey	97
Arthur P. Davis	70
Logan W. Page	1
John E. Greiner	1
Arthur W. Thompson	1
Charles O. Vandevanter	1
A. C. Shand*	1

*Not a member.

A MEMBER.—I move that Mr. Thomas Earle be selected as a member of the Nominating Committee from District No. 4.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Thomas Earle be a member of the Nominating Committee from District No. 4. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—District No. 5: Total number of suggestions received, 246, as follows:

John W. Woermann	66
John W. Alvord	60
William L. Breckinridge	59
Alexander Maitland, Jr.	46
Webster Gazlay	14
John V. Hanna	1

A MEMBER.—I move that Mr. John W. Woermann be named as a member of the Nominating Committee from District No. 5.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. John W. Woermann be a member of the Nominating Committee from District No. 5. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—District No. 6: Total number of suggestions received, 223, as follows:

Thomas U. Taylor	69
James N. Hazlehurst	66
John F. Coleman	52
Walton P. Darwin	22
Paul H. Norcross	11
Harry O. Cole	1
Howard N. Eavenson	1
James C. Nagle	1

A MEMBER.—I move that Mr. Thomas U. Taylor be named as a member of the Nominating Committee from District No. 6.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Thomas U. Taylor be a member of the Nominating Committee from District No. 6. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—District No. 7: Total number of suggestions received, 287, as follows:

Milo S. Ketchum	123
William C. Hammatt	96
Clifford S. MacCalla	61

Nominating Committee (continued).

Ralph M. Conner	1
Homer Hamlin	1
Frank C. Horn	1
Joseph B. Lippincott	1
William Mulholland	1
Charles D. Vail	1
R. H. Thomson	1

A MEMBER.—It is moved that Mr. Milo S. Ketchum be named as a member of the Nominating Committee from District No. 7.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Milo S. Ketchum be a member of the Nominating Committee from District No. 7. Those in favor of the motion say "aye"; contrary-minded, "no". It is carried.

Report of Committee on Valuation of Public Utilities.

The next business in order is the Report of the Special Committee on the Valuation of Public Utilities, Frederic P. Stearns, Chairman.

MR. STEARNS.—Mr. President and members of the Society. The report of the Special Committee on the Valuation of Public Utilities is signed by six of the seven members. Mr. Henry M. Byllesby, who is a member of the Committee was unable to take any part whatever in its deliberations, and said that he was even unable to read the drafts that had been sent to him, and he therefore requested that his name be omitted. That is not a case of opposition on his part, so far as he has expressed himself. This report covers a good many fields of an intricate subject, and it would be useless to think of reading it or any part of it. Similarly, I believe, it would be useless in a meeting of this sort to discuss it; and at the suggestion of some members, and in accordance with my own views, if the report is before the Society, I would like to offer this motion.

THE PRESIDENT.—Please read it.

MR. STEARNS.—"*Resolved,* That the Progress Report of this Committee, together with all discussion thereon up to September 1st, 1914, or to such later date as the Board of Direction may fix, be referred back to the Committee for further consideration; and that in the meantime the Board of Direction be requested to assign a date for the written and oral discussion of this subject."

MR. WILLIAMS.—I second the motion.

THE PRESIDENT.—It is moved and seconded that this be passed. Is there anybody who wishes it re-read? It is moved that this progress report, together with the written discussion, be referred back to the Committee, and that in the meantime the Board of Direction assign a date for the written and oral discussion on the subject.

A MEMBER.—I second the motion.

THE PRESIDENT.—It has been seconded. Is there any discussion?

Those in favor of the motion please say "aye"; contrary-minded, "no". It is carried.

THE SECRETARY.—As a matter of information I should like to ask whether it is the intention of this resolution not to publish any of this discussion. Our usual method would be to publish it as it came in, in *Proceedings*. It was not intended to shut that off?

MR. STEARNS.—Not in the least. I suppose that written discussions would be published from time to time, so that all the members of the Society would have the benefit of it. The idea was to initiate a date a considerable way in the future, so as to give ample time for the discussion of this important subject, and that the Committee might have the advantage of these discussions so that it might amend the report if it saw fit to do so.

A MEMBER.—Mr. President, I move that inasmuch as each member of the Society has a copy of that report, that it be not published in the *Proceedings* until the final report of the Committee is issued.

Motion duly seconded.

THE PRESIDENT.—It is moved and seconded that, inasmuch as every member of the Society now has in his possession a copy of this report, it be not published until it is published in the form finally arrived at by the Committee. Is there any discussion?

J. N. CHESTER, M. AM. SOC. C. E.—May I add to that, that copies of this report shall not be given out to parties not members of the Society?

THE PRESIDENT.—That can be a later motion.

A MEMBER.—That can be added to the motion, if the original maker agrees.

THE PRESIDENT.—Will you please state that again?

MR. CHESTER.—That copies of this report shall not be given out, so far as it is in the power of the Society and its officers to prevent it, to parties not members of the Society; that no parties not members of the Society shall be furnished with a copy of the report.

THE PRESIDENT.—You mean by the Society?

MR. CHESTER.—By the Society or its officers or members.

THE PRESIDENT.—That amendment is accepted.

MR. WILLIAMS.—Mr. President, it seems to me that this resolution is one of questionable policy. In the first place, I suppose a large number will have mislaid the report. I think it should be published in the *Proceedings*. I think there is nothing in here that should be kept secret. It is a progress report. We want the members of this Society to discuss it, and I think we want persons who are not members of this Society to discuss it, too. I do not believe that the American Society of Civil Engineers wants to keep under the bushel the work that it is doing, even during its progress, and I sincerely hope this motion will not prevail.

Report of Committee on Valuation (continued).

A Member.—While I agree with the motion—my remarks have been brought out by finding this report in a Public Service Commissioner's hands, and he was using it on the supposition that it was the actual report of this Committee, not a progress report at all. Now, if what Mr. Williams says are the sentiments of the meeting, let it be made clear that this is only a progress report, subject to further amendment before it is adopted.

The President.—The motion before the meeting is that, inasmuch as every member of the Society has received a copy of this report, it be not printed in the *Proceedings,* until it is presented in its final form by the Committee.

A Member.—I agree with Mr. Williams thoroughly, and, as a member of the Committee, I feel that all of us should have the benefit of everybody's comments on this subject. There is no objection whatever to making it evident that it is only a progress report, and that it has not been accepted in any way by the Society, if that can be made plain; but we do not want to be tied up, as Mr. Williams said, and hide our work under a bushel. It should be made public to everybody, the attorneys who deal with this subject should give us the benefit of their judgment upon it.

The President.—Are you ready for the motion?

Mr. Stearns.—I move a division of that question. I am not opposed to deferring its publication in the *Proceedings,* although I think it may be questionable whether that is desirable, but the other motion, that no report shall be sent out to anybody outside of the membership of the Society, I think is very unwise. There are some people from whom we wish to get discussions, and the report has already been sent to them. There are some to whom we wish to give the benefit of what the Committee has done.

Mr. Chester.—I will withdraw that, under the conditions stated, that it is a progress report, and not absolutely final.

Mr. Stearns.—I have nothing to say, if that part of the motion is withdrawn.

The President.—It is withdrawn. The motion is that it be not printed in the *Proceedings* until it is printed in the proper form. Those in favor say "aye"; contrary-minded, "no". Carried.

Mr. Wallace.—If it would be in order, I would like it if the Committee would explain why, in the valuation of railway property, it heads its report "For the purposes of rate-making"?

The President.—Will Mr. Stearns answer that question?

Mr. Stearns.—Mr. President, the Committee is not a Committee for the valuation of railway properties; in accordance with its title, it is for railway property and other public utilities. The amount of work involved in this report was enormous. It seemed as if we would never get it out, and so, as we believed that valuation for purchase and

sale, valuation for taxation, and valuation for other questions were not identically the same, we decided to limit this report to one side of the question, namely, the valuation for the purposes of rate-making. That is the most comprehensive kind of valuation, and brings up the greatest number of questions, so that we thought we would do the most good by taking up that branch of the subject first.

A MEMBER.—While there may be some question of doubt as to the different purposes that underlie these matters of valuation, I think there can be no doubt whatever in the minds of every member of the Society who has seen the preliminary report as to the enormous amount of work that the Special Committee has done, and I think that the Society owes to the Committee an appreciation of the tremendous amount of work it has done.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that the thanks of the Society be extended to this Committee for the very large amount of very valuable work it has done on this report. All in favor of the motion please say "aye"; contrary-minded, "no". Carried.

Report of Committee on Bituminous Materials for Road Construction.

The next business is the report of the Special Committee on Bituminous Materials for Road Construction, W. W. Crosby, Chairman.

W. W. CROSBY, M. AM. SOC. C. E.—Mr. President, as the report of the Committee has been printed, and is rather long, I would like to present the report by title, and if Mr. Stearns will allow me to profit by the resolution which he introduced, by the resolution in support of his Committee, I would like to offer the same resolution.

THE PRESIDENT.—You have heard the motion by the Chairman of the Committee that the same action that was taken with reference to the previous report be taken with reference to this report. Is the motion seconded?

Motion duly seconded.

THE PRESIDENT.—Are you ready for the question?

MR. WHINERY.—Mr. President, inasmuch as this Committee has been in existence for five years, and has rendered annual reports, I think, inasmuch as these committees are usually appointed to get a definite answer or a definite opinion on certain subjects at the time, and as I believe that the continued existence for years and years, of these special committees, is not in the best interests of the Society, I move that this Committee be continued and directed to submit a final report at the next Annual Meeting.

THE PRESIDENT.—A final report, do you mean?

MR. WHINERY.—A final report.

THE PRESIDENT.—Mr. Whinery moves that the Committee be continued for the present, but be directed to bring in a final report at the next Annual Meeting. Is the motion seconded?

Motion duly seconded.

Report of Committee on Bituminous Materials (continued).

Mr. Crosby.—Much as I dislike to make a point of order against Mr. Whinery, I believe there is a resolution before the House which antedates his. I should like to say a word on Mr. Whinery's motion when the proper time arrives, but having made the point of order I want a ruling on it.

The President.—The gentleman's point of order is correct. The motion that the same action be taken as was taken with reference to the previous report, was not decided. Those in favor of taking the same action on this report that was taken with respect to the previous report say "aye"; contrary-minded, "no". Carried.

Now Mr. Whinery's motion is that the Committee be continued for a year, but requested to bring in a final report at the next Annual Meeting.

Mr. Crosby.—The Committee has fully realized from the beginning the force of what Mr. Whinery has stated, and has endeavored to bring this matter to a conclusion; but, in the present state of this subject, it has found it extremely difficult each year to present a final report on many of the questions. The Committee intended during the coming year to work even harder, in order to bring in a final report. It is rather difficult to foresee whether that is possible or not, as the Committee has recently been in communication with the Board of Direction, I presume partly with the same purpose in view. I think that it might be wise not to discourage or to limit the activities of the Committee by directing that their energies cease during the coming year.

There are many questions which are now being investigated, and which will take time for settlement; and though it is possible that all of this will be completed within the coming year, to my mind it is hardly probable. I simply wish to make that statement in connection with Mr. Whinery's motion.

The President.—Are you ready for the question? The motion was duly made and seconded? Are you ready for the question? The motion is that the Committee be continued for a year and requested to bring in a final report at the next Annual Meeting. Those in favor of the motion say "aye"; contrary-minded, "no". The chair believes it is not carried. Do you wish a showing of the hands, Mr. Whinery? Those in favor of the motion please raise the hand.

The Secretary.—Twenty-six.

The President.—Those opposed. It is not carried.

A Member.—What is the status of the Committee? Is it discontinued?

The President.—The Committee continues. The Committee is in existence, but has not been directed to bring in a final report next year as was the purpose of the motion.

A Member.—I am not quite clear whether Mr. Crosby's motion included a vote of thanks to the Committee for its work? If not, I

move that a vote of thanks be extended to the Committee for its work.

Motion seconded.

THE PRESIDENT.—It is moved and seconded that the thanks of the Society be extended to this Committee for its work in preparing this report. Those in favor of the motion please say "aye"; contrary-minded, "no". Carried.

Report of Committee on Employment and Compensation of Civil Engineers.

The next is the Report of the Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers, Alfred Noble, Chairman. Will Professor Jackson present the report.

MR. JACKSON.—I refused to sign the report, although I think it is a very valuable one, and the investigation is a very valuable one; but I refused to sign it for the reason that the average presented was a too optimistic statement of the circumstances under which engineers work, and for the reason that the minimum presented so pessimistic a situation that we ought to investigate that part of it. I therefore move that Mr. Stearn's motion be applied to the report of this Committee, and that the report be not printed in the *Proceedings* or *Transactions* until the Committee has had an opportunity to investigate part of it, and has made a final report.

Motion duly seconded.

THE PRESIDENT.—You heard the motion. I presume it is not necessary to have it repeated. Those in favor say "aye"; contrary-minded, "no". It is carried.

Report of Committee on Columns and Struts.

THE PRESIDENT.—The report of the Committee on Steel Columns and Struts, A. L. Bowman, Chairman. In order that the speakers may be heard, it is very desirable that we preserve quiet as far as possible.

A. L. BOWMAN, M. AM. SOC. C. E.—This report is under date of December 15th, 1913.

DECEMBER 15TH, 1913.

The Special Committee, "to consider and report upon the design, ultimate strength, and safe working values of Steel Columns and Struts," presents the following report of progress:

The proposed programme of column tests which is being made for the Society by the Bureau of Standards of the U. S. Government, S. W. Stratton, Director, has been somewhat delayed owing to the rigid specifications and the pressure of business in the steel mills, but the Committee is glad to report that seventy columns have been fabricated and delivered in Washington to date, and that some twenty of these have already been tested. The testing is being done on the new 2 300 000-lb. testing machine which has recently been built for the Bureau of Standards at Washington.

From conferences with the sub-committee on Iron and Steel Structures, of the American Railway Engineering Association, Mr. W. H.

Report of Committee on Columns and Struts (continued).

Moore, Chairman, we learn that the Bureau of Standards has in progress for them a correlated programme of steel column tests. These tests will assist materially in interpreting the results of our own programme.

At the October meeting of the Board of Direction, Mr. James H. Edwards was appointed a member of the Committee, to fill the vacancy caused by the death of Mr. Alfred P. Boller.

Awaiting the completion of the tests now under way, your Committee makes this progress report.

Committee:	For the Committee:
Austin Lord Bowman,	Austin Lord Bowman, *Chairman.*
J. H. Edwards,	
E. Gerber,	Lewis D. Rights, *Secretary.*
Chas. F. Loweth,	
Ralph Modjeski,	
Frank C. Osborn,	
Geo. H. Pegram,	
Lewis D. Rights,	
Geo. F. Swain,	
Emil Swensson,	
J. R. Worcester.	

Mr. Bowman.—I might state that these columns are now being tested by the Bureau of Standards, at Washington, D. C.

A Member.—I move that the report be received and placed on file.

Motion seconded.

The President.—It is moved that the report be received and placed on file. It has not been previously presented. Those in favor of the motion say "aye"; contrary-minded, "no". Carried.

Report of Committee on Bearing Value of Soils.

The Report of the Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, Robert A. Cummings, Chairman.

Mr. Cummings presented the following:

Jan. 20th, 1914.

To The American Society of Civil Engineers:

Your Special Committee appointed by the Board of Direction "to codify present practice on the bearing value of soils and to report upon the physical characteristics of soils in relation to engineering structures" respectfully submits the following Report of Progress.

Since its appointment your Committee has given careful thought to its subject. In the first place, a schedule of questions and a circular letter have been prepared inviting co-operation by those possess-

ing information concerning tests and data from existing structures and customary local practice. The preparation of a bibliography has also been commenced.

Regarding the second phase of the subject, your Committee recognizes that the classifying and correlating of the soils of the United States is of the utmost importance, and must be its first consideration. To this end a great deal of time has been taken up by the examination of technical literature. This is expected to furnish the basis for a satisfactory classification.

Your Committee has received assurances from U. S. Bureau of Standards and U. S. Bureau of Mines, of their active interest and offer of co-operation of these branches of the Federal Government in a scientific investigation of the physics of soils.

In consequence of the importance of the subject and the incompleteness of the data on hand your Committee can do no more than report progress at this time and requests its continuance.

Robert A. Cummings, *Chairman;*
W. J. Douglas,
J. C. Meem,
Samuel Tobias Wagner,
E. C. Shankland,
F. M. Kerr.

The President.—You have heard the report of the Committee. What is your pleasure?

A Member.—I move that it be received and placed on file.

Motion duly seconded.

The President.—Gentlemen, you have heard the motion. Those in favor please say "aye"; contrary-minded, "no". Carried.

The Report from the Special Committee on Concrete and Reinforced Concrete, Joseph R. Worcester, Chairman.

Report of Committee on Concrete and Reinforced Concrete.

Richard L. Humphrey, M. Am. Soc. C. E.—Mr. Chairman, Mr. Worcester is not here.

"Philadelphia, Penn., January, 1914.

"*To the President & Members of the*
American Society of Civil Engineers,
220 West Fifty-seventh St., New York.

"Your Special Committee on Concrete and Reinforced Concrete has nothing at present to add to the report which it presented at the last annual meeting. While it is not complete, there being a number of developing points or details upon which a further report may be desirable and to which your Committee is giving consideration, it is not likely that it will have anything in shape to present to the Society for some considerable time. Your Committee would recommend that this report be printed in the *Transactions* so as to reach many who do not receive or keep the *Proceedings.*

Report of Committee on Concrete and Reinforced Concrete (continued).

"In view of the fact that matters connected with the art are now developing and under consideration, your Committee suggests that it be continued, with the understanding that it will not be expected to present a report until it has reached further conclusions.

"Respectfully submitted in behalf of the Committee.

"J. R. Worcester,
"Chairman,
"Richard L. Humphrey,
"Secretary."

The President.—You have heard the report of the Committee, gentlemen. What is your pleasure?

A Member.—I move that the report be received and placed on file, and the Committee continued.

Motion seconded.

The President.—That would be done automatically. Those in favor please say "aye"; contrary-minded, "no". Carried.

What is your pleasure with reference to the recommendation of the report, that the report of the Committee be printed in the *Transactions?* The Committee recommends that a previous report, as I understand it, this previous full report be printed in the *Transactions,* the report presented at the last Annual Meeting.

A Member.—I make that motion.

Motion duly seconded.

The President.—It is moved and seconded that the report of this Committee at the last Annual Meeting, which was a very full report, be printed in the *Transactions,* as recommended by the Committee today. Any discussion? All those in favor of the motion please say "aye"; contrary-minded, "no". Carried.

Report of Committee on Engineering Education.

The report of the Special Committee on Engineering Education, Desmond FitzGerald, Chairman.

Mr. FitzGerald.—Mr. Chairman and Gentlemen: I had the honor of presenting a verbal report on the subject of Engineering Education. As the time is passing rapidly, and as this is a very important subject, I should like to have the opportunity of amplifying it to a certain extent later, but I will give you the gist of the position of the Committee at the present time. For the last two years we have been endeavoring to get the Carnegie Foundation to begin its long promised work in the matter of making a thorough study of engineering education, and we have been very much disappointed, Mr. President, to find that at the end of two years nothing could be done.

Feeling that this Society especially was anxious that something should be done, or some explanation given, the Committee prepared a carefully written report on the subject of Engineering Education, giving their own views, the views which they had arrived at after five years of careful study and investigation, as far as possible, on this important subject.

This written report, Mr. President, covers almost all the debatable grounds which have been brought up within the last few years, in regard to the methods of treating engineering education. It treated of the matter of discipline, and recognized the importance of that portion of engineering education. It treated of the matter of special courses; it treated of the matter of physical training, of the introduction of more English, of the graduation of the student with a better knowledge of his own counts. It treated comprehensively, as far as it could, of those subjects, and now at the last moment it has been determined by the Committee not to present this report. Why? Because since January 2d, last, the whole matter has changed.

The clouds that obscured the sky have passed away. The Carnegie Foundation have explained why they have not undertaken the work in the last two years, and they have given us written promises, written notes, stating that they will begin just as soon as they can organize the office and find the proper man to whom to entrust the important work. I hold in my hand a copy of the resolution of that important meeting, which took place here in New York on January 2d, and that will explain to you why—knowing that that committee meeting was coming off—that will explain to you why we held our own report back, and why we finally determined to await the result of a more important investigation.

THE PRESIDENT.—Will those in the rear be so good as to maintain quiet as far as possible, so that the report may be heard.

MR. FITZGERALD.—I do not intend to burden you with reading the whole of this report, but I should like to say that the Carnegie Foundation for the advancement of teaching have voted as follows:

"*First.*—The Board of Directors of the Foundation has authorized the making of a thorough study of engineering education;

"*Second.*—The Foundation will pay all expenses attendant upon making the study;

"*Third.*—The study will be under the supervision of the Foundation in consultation with this Committee, this Committee naming not only the Committee that now appears before you as the Committee of this Society, but also committees of all the other engineering societies, and all societies for the promotion of engineering education;

"*Fourth.*—The report embodied the result of the studies that will be published by the Foundation, and final decision with reference to matters to be included therein will rest with the Foundation."

The rest of the resolution treats of the methods of conducting the work. You will see, gentlemen, from this, that although it may be true, as has been stated here, this is a period of restlessness, a period of change, yet that does not imply that change in itself means retrogression. Who is there of us that believes all within the great maelstrom that is going on at the present day, every change is not going to produce progress, that men are not going to be benefited by that change?

Report of Committee on Engineering Education (continued).

Almost any current is better than stagnation, and we feel as a Committee that this work now, this important work of making a thorough study in the schools themselves and of studying the results and judging the results by the character, the abilities, the attainments, and the work of the graduates of these schools, will result probably in bringing before the schools some method of improving methods of study.

This may be true. We hope devoutly that it will be true, that it will come about in that way. Now, gentlemen, however important all of our own personal interests at the present day may appear to us, there is little doubt in my mind, and I believe in the minds of the committee that has been studying this matter, that we are on the eve of the greatest change in engineering education. We believe that the foundations of that structure have been laid so deep and that they have already attained such strength and power that they are going to produce in the very near future a structure before which all other methods of education must bow their heads in acknowledgment of superiority.

There is no doubt in the minds of our Committee, I think, that engineering education is advancing by such leaps and bounds that in a very short time, comparatively, it will absorb such a large amount of the scientific, literary, and intellectual products of thought of this century that it will be the wonder of the world.

The education for theology, for jurisprudence, for the sciences, will be almost a pigmy in comparison, and we, as engineers, I think, should feel some of the responsibility that goes with that enormous progress and development. I ask you, gentlemen, now to continue this Committee in order that we can aid the Carnegie Foundation by consultations, for a year or two longer, until we can watch their results.

A Member.—I move that the report be received and filed.

Motion seconded.

The President.—You have heard the remarks of Mr. FitzGerald. The Committee presents no formal report, but will be automatically continued.

W. de H. Washington, Assoc. M. Am. Soc. C. E.—I think, Mr. Chairman, that a vote of thanks for the important work of this Committee will be certainly due to them.

Motion duly seconded.

The President.—It is moved and seconded that the thanks of the Society be extended to this Committee. Any discussion? Those in favor will please say "aye"; contrary-minded, "no". Carried.

Reports from the Board of Direction by the Secretary. What has the Secretary to present?

Award of Prizes.

The Secretary.—I am instructed by the Board of Direction to state that Messrs. Charles M. Spofford, Daniel W. Mead, and Walter Loring Webb, were appointed a Committee to Recommend the Award

of Prizes for the year ending July, 1913. That after this Committee had reported, the Board of Direction has awarded the Prizes as follows:

THE NORMAN MEDAL to Paper No. 1235 entitled "Air Resistances to Trains in Tube Tunnels" by J. V. Davies, M. Am. Soc. C. E.

THE THOMAS FITCH ROWLAND PRIZE to Paper No. 1231 entitled "The Laramie-Poudre Tunnel" by Burgis G. Coy, Assoc. M. Am. Soc. C. E.

THE J. JAMES R. CROES MEDAL to Paper No. 1222 entitled "The Problem of the Lower West Side Manhattan Water-Front of the Port of New York" by B. F. Cresson, Jr., M. Am. Soc. C. E.

THE JAMES LAURIE PRIZE to Paper No. 1218 entitled "Construction of the Morena Rock Fill Dam, San Diego County, California" by M. M. O'Shaughnessy, M. Am. Soc. C. E.

THE PRESIDENT.—This report of the Board of Direction is simply presented for the information of the Society.

THE SECRETARY.—At the June 4th, 1913, Meeting of the Society, the following Resolutions were passed:

Special Committee on Stresses in Rails, Ties, etc.

"That it is the sense of this meeting that the American Society of Civil Engineers should appoint a Special Committee to act jointly with the American Railway Engineering Association in studying and experimenting on the stresses in railroad rails, ties, etc.", and

"That the matter of appointing this Committee be left with the Board of Direction with full power."

I have the honor to state that the following Committee to report on this subject has been appointed by the Board:

A. N. TALBOT, *Chairman*, A. S. BALDWIN, J. B. BERRY, G. H. BREMNER, JOHN BRUNNER, W. J. BURTON, CHARLES S. CHURCHILL, W. C. CUSHING, E. GERBER, ROBERT W. HUNT, GEORGE W. KITTREDGE, WILLIAM MCNAB, G. J. RAY, F. E. TURNEAURE, J. E. WILLOUGHBY.

THE SECRETARY.—I am instructed by the Board to report the following:

On July 2d, 1913, Augustus Smith, M. Am. Soc. C. E., addressed a communication to the President, which he in turn presented to the Board of Direction for consideration.

"BAYONNE, N. J.,
"JULY 2ND, 1913.

"PROF. GEORGE F. SWAIN,
"Pres. American Soc. C. E.,
"Harvard Univ., Boston, Mass.

Proposed Appointment of an Arbitration Board.

"Dear Sir:—I have read, with extreme satisfaction, your Annual Address and regret that I was prevented from hearing it at Ottawa.

"Engineers and the American Society of Civil Engineers have the greatest opportunities open to them in public work for Engineering

Proposed Appointment of an Arbitration Board (continued).

involves Leadership in every sense. The Engineer has to deal with nature in making his plans and with man in the execution thereof. Until an Engineer is equally at home when weighing the forces of nature or the motives of men, he has not reached his full growth.

"Encouraged by views so well expressed in your address, I am writing to obtain your opinion of the practicability of a vision that I have long cherished that the American Society of Civil Engineers would establish a Judiciary Department for adjudicating disputes arising in engineering construction.

"My idea, roughly, is to create an Arbitration Board of 50 to 100 men to which corporate members would be elected to fill vacancies caused by death or retirement, on the regular letter ballot, with the other Officers of the Society, when vacancies in the Arbitration Board existed. The Board would be filled by a certain number of engineers from each District.

"The services of this Board would be available to any disputants making proper application to the Society therefor, whether or not said disputants were represented by Membership in the Society.

"A Bench of three members of the Arbitration Board would be assigned to hear and judge of each such case; the selection of the particular three judges assigned to a given case being made by some power created within the Arbitration Board. The disputants would have no voice in the selection of the judges.

"A schedule of fixed fees depending on the number of hearings, or the number of witnesses or the quantity of evidence, or, perhaps, on the amount involved, would be provided to compensate the arbiters assigned to a case.

"I see the following advantages of such a Board:

"1. It would provide a Court competent to hear and in sympathy with the matters of dispute that arise in construction work. No jury would be necessary. Few 'expert' witnesses would be called.

"2. The delay and much of the expense incident to invoking the State or U. S. Courts will be eliminated.

"3. The expense and vexatious delays and the technicalities that seem to be inevitable in an actual trial in the established Courts would be greatly reduced.

"4. The difficulty of obtaining 'Arbitrators' where each side selects his own Arbitrators or where both sides try to agree on one Arbitrator will be done away with.

"5. The money paid for such litigation will be kept in the profession.

"6. An election to the Arbitration Board will be an honor second only to that of President of the Society, to be looked forward to by the older members of the Society. As the Society increases in numbers, there will be a growing need of additional Society Honors for, on numerical reasons, few can possibly be rewarded by election to the Presidency.

"7. An Arbitration Board would increase the power and prestige of the Society and would expand its usefulness to the public.

"It might be urged that the decisions of such a Board could not be enforced. I do not think that this will nullify the usefulness of its work for the following reasons.

"1. Decisions of all of the Public Courts, except the Supreme Court, are not final but can usually be appealed. Yet, how few cases are carried to the Supreme Court.

"2. Most disputes that cannot be settled by the parties themselves arise from a sense of personal injury or slight. The disputants get mad. They must get a decision from an outside source. I believe that an intelligent sympathetic opinion from an impartial bench of competent men in their line of business in whose selection they had no part will go further to give 'satisfaction' than the usual Court Decision which frequently betrays lack of appreciation of the essentials involved and too often is a mere compromise.

"3. Better minds would be on such a Bench as the Arbitration Board would provide than are usually found on the Court Benches, to which Judges are too often elected or appointed by questionable methods. Few disputants are so bitter that they will not listen to 'Reason' when Reason really talks.

"4. Few would care to risk the unpopularity that would result by disregarding the decision of a Bench of the Arbitration Board of the American Society of Civil Engineers.

"5. There are successful precedents for such a Board in the Board of Governors of the New York Stock Exchange; in the Arbitration Committee of the New York Chamber of Commerce; in the Court-martials of the Army and of the Navy; in the Regatta Committee of any Yacht Club and in the By-Laws of nearly every Club or fraternity.

"I hope that you will consider such a Board within the sphere of usefulness of the American Society of Civil Engineers and will call on me to do whatever you think I can do towards promoting its organization forthwith.

"Yours respectfully,

"AUGUSTUS SMITH."

THE SECRETARY.—I am instructed by the Board to report to the Society that, having given this subject careful consideration, the following was adopted by the Board at its meeting of December 3d, 1913:

"The Board of Direction of the American Society of Civil Engineers has considered the communication of Mr. Augustus Smith concerning the appointment of an Arbitration Board, and has extensively canvassed the subject with members of the Society in various localities, some of whom have had considerable experience in arbitration matters.

"After considering the many views presented, the Board is of the opinion that Arbitration of honest differences is to be recommended in preference to legal process as much as possible.

"That, however, an Arbitration Board as proposed by Mr. Smith would not be practical at the present time, nor until arbitration is more extensively practised, because of the difficulty of devising a satisfactory method of selecting competent men to membership of such a Board; further, because of the impracticability of so constituting the Board of Arbitration that it may have experts in all lines so distributed that

Proposed Appointment of an Arbitration Board (continued).

men appointed on any given case need not be drawn from too great a distance, and further, because of the difficulty of assigning men from such a Board to the satisfaction of the disputants who may prefer persons better known to themselves.

That is all I have from the Board, sir. I have another communication that I will read, a letter from W. C. Sawyer, Assoc. M. Am. Soc. C. E.

"626 So. Hope St., Los Angeles, Cal.
Dec. 26, '13.

"Mr. Chas. Warren Hunt,
"Sec'y Am. Soc. Civ. Engr's.

Local Associations, etc.

"Dear Sir:—I respectfully suggest that you bring before the members at the Annual Meeting for discussion the matter of changing the name of the local associations to something shorter.

"Allow me also to suggest that you discuss at that meeting the possibility of refunding to local associations 10% to 25% of the dues received from such members as are affiliated with such associations.

"As an alternative, though less desirable from our viewpoint, the dues might be raised $5.00 applicable to all members living within one hundred miles of the headquarters of a local branch.

"The members of all grades living within such circle should thereby become active members of the local associations. The members might be given opportunity to pay the five dollars direct to the local treasurer of the local association, but upon failure to do so they should be collected by the parent society under usual conditions and refunded. This would greatly strengthen the local organizations. Of course a certain number of applicants should be necessary before such an association could be formed under such conditions. It might also be optional upon the part of local associations whether they would adopt such conditions.

"These are my personal suggestions which have not been acted upon by our association.

"We were all very much disappointed at missing a visit from you but enjoyed having Mr. Metcalf present. Our Annual Meeting will be held Jan. 5, '14, at which time the results of letter ballot election of permanent officers will be announced.

"Yours truly,

"W. C. Sawyer, *Temp. Sec'y.*

The President.—The Chairman must insist that there be quiet at the rear of the room. You have heard this communication that the writer desires to have brought before the Society? What is your pleasure?

James Owen, M. Am. Soc. C. E.—I move that it be referred to the Board of Direction with power.

Motion seconded.

The President.—Is there any discussion on the question? Those in favor of the motion please say "aye"; contrary-minded, "no". The motion is carried.

The Secretary informs me that the Report of the Tellers is not quite ready, and will not be ready for some ten minutes. You may, therefore, introduce any new business if any member desires to do so.

C. H. HIGGINS, M. AM. SOC. C. E.—Mr. President, I have a motion to introduce, the subject matter of which has been so widely discussed that no introduction is necessary. I move that the Board of Direction be and hereby is directed to submit to the membership by letter-ballot before the Annual Convention the following resolution:

"*Resolved,* That the American Society of Civil Engineers believes it to be in the public interest that the practice of engineering be regulated by statute."

Resolution Relating to Regulation of Practice of Engineering.

I submit this motion.

THE PRESIDENT.—Gentlemen, you have heard the motion. Is it seconded?

Motion duly seconded.

THE PRESIDENT.—The motion has been seconded. Any discussion of the motion? The motion is that the Board of Direction submit to the membership by letter-ballot the question which the member will please read once more.

MR. HIGGINS.—"*Resolved,* That the American Society of Civil Engineers believes it to be in the public interest that the practice of engineering be regulated by statute."

A MEMBER.—That that be the sense of the meeting? I believe that the form would have to be changed to be legal.

THE PRESIDENT.—The Secretary suggests that perhaps you do not understand the motion, which is, that the Board of Direction send out this question to ballot, so that the members can express themselves. Please read the resolution once more.

MR. HIGGINS.—"*Resolved,* That the American Society of Civil Engineers believes it to be in the public interest that the practice of engineering be regulated by statute."

THE PRESIDENT.—The motion is that the Board of Direction be, and hereby is directed to submit to the membership by letter-ballot the resolution already read, and that it be so submitted before the next Annual Convention; and the members will have an opportunity to vote aye or no in regard to the motion.

MR. STEARNS.—Mr. Chairman, I hope that this motion will not pass. It has not been before any committee or before the Board of Direction, and I do not think we should take such summary action.

THE PRESIDENT.—Any further discussion?

L. C. WASON, M. AM. SOC. C. E.—Mr. President, I would like to suggest that the statutes of every State differ from one another, and it would lead to endless trouble. If there is any recommendation at all, I should think it would be for the National Government to enact

Regulation of Practice of Engineering (continued).

such legislation, so that it would be uniform throughout the United States; and, if the motion is carried, it should have some such rider as that.

A MEMBER.—Mr. Chairman, I move that the motion be laid on the table.

Motion duly seconded.

THE PRESIDENT.—It is moved and seconded that the motion be laid on the table. Those in favor of the motion please say "aye"; contrary-minded, "no". Carried.

F. H. NEWELL, M. AM. SOC. C. E.—The Special Committee on A National Water Law, composed of members in various parts of the United States, has not yet been fully organized, but, if it is appropriate, I will ask that the President request that the members who are here meet after the meeting.

THE PRESIDENT.—The Chairman of the Committee on A National Water Law, recently appointed by the Board of Direction, requests that all members here meet with him in the Secretary's office, or in the office opposite the Secretary's office, at the adjournment of this meeting. Is there any other new business?

THE SECRETARY.—Mr. President, in connection with the resolution which was just offered and laid upon the table, I think it may be of interest to the members of the Society to know that a Joint Committee composed of representatives from each of the five National Engineering Societies has been instructed—that the representatives of each have been instructed—by their several Boards to prepare the draft of a bill for the registration of engineers.

The primary object of this work was to head off vicious legislation which has appeared in one or two States. In a secondary way we know that such legislation is pending, and it is the intention to produce a draft of a bill which will be unobjectionable at all events to the Profession; and a great many believe that if such a bill were adopted in a State like New York that other States would follow very soon. As a matter of fact, the State of Pennsylvania has already appointed a Commission of three engineers to study the subject and report next November. That bill, as I happen to be on that committee, I know is practically ready for reporting back to the governing bodies of each of these five National Engineering Societies, and should it receive the endorsement of the governing bodies of those societies I think it will come pretty near becoming a law.

W. G. FEDERLEIN, ASSOC. M. AM. SOC. C. E.—Regarding the proposed bill, what has been stated may be true, but this Society has never expressed its opinion upon the question of whether the members want legislation of that kind or not. I therefore make a motion that the motion made by Mr. Higgins be taken off the table and reconsidered.

THE PRESIDENT.—It is moved that the motion of Mr. Higgins, which was laid on the table, be taken from the table. Is the motion seconded?

MR. WILLIAMS.—I rise to a point of order. There has been no business transacted by this body since the making of that motion, and therefore the motion is not in order.

THE PRESIDENT.—The motion has not been seconded. The Secretary now has the report of the Tellers. You will please listen to the report.

THE SECRETARY.—The Tellers appointed to canvass the ballots for officers of the Society for 1914 report as follows: Ballot for Officers.

Total number of ballots received		1 729
Ballots without signature	19	
" stamped, not signed	5	
" from members in arrears of dues	6	
Total number not entitled to vote		30
Ballots canvassed		1 699

For President:

HUNTER McDONALD	1 694
Scattering	5

For Vice-Presidents:

CHARLES F. LOWETH	1 678
GARDNER S. WILLIAMS	1 668
Scattering	18

For Treasurer:

JOHN F. WALLACE	1 693
Scattering	4

For Directors:

District No. 1.	GEORGE W. FULLER	1 645
	ARTHUR S. TUTTLE	1 631
	Scattering	9
District No. 2.	CHARLES H. KEEFER	1 638
	Scattering	2
District No. 3.	MORTIMER E. COOLEY	1 638
	EUGENE E. HASKELL	1 634
	Scattering	16
District No. 5.	RICHARD MONTFORT	1 639
	Scattering	9

Election of Officers.

THE PRESIDENT.—Gentlemen, you have heard the report of the Tellers. I would therefore declare that Mr. Hunter McDonald has been elected your President for the coming year, and that you have elected as Vice-Presidents Mr. Charles F. Loweth and Mr. Gardner S. Williams; for Treasurer Mr. John F. Wallace; for Directors from District No. 1, Messrs. George W. Fuller and Arthur S. Tuttle; from District No. 2, Mr. Charles H. Keefer; from District No. 3, Messrs. Mortimer E. Cooley and Eugene E. Haskell; from District No. 5, Mr. Richard Montfort.

The time has now come, gentlemen, for me to lay down the duties which you imposed upon me a year ago and to introduce to you my successor in office; and in doing so I wish to express to the membership of the Society one and all, the deep sense of the honor you did me a year ago and my thanks for the courtesy and kindness which you have shown me during the year, notwithstanding my many deficiences.

I will ask Mr. Ockerson and Admiral Endicott to escort the President-elect to the chair.

Remarks by President McDonald.

HUNTER McDONALD, PRESIDENT, AM. SOC. C. E.—Fellow members of the American Society of Civil Engineers, the Secretary has suggested to me that a few extemporaneous remarks might be proper on this occasion, but anticipating that the temper of this meeting might be somewhat belligerent in view of the discussion which I knew was coming, I thought it more prudent to reduce my remarks to writing, and if you will excuse me I will present them.

I once heard a distinguished lawyer addressing a body of engineers declare that the members of the profession were paid less than any person he knew in proportion to the value of their services and the time and expense required to properly equip them for their work. He said that the only way he could account for their zeal and devotion to their profession was that he thought they enjoyed the chase more than the killing. All of us are in search of and enjoy to the fullest extent the honor and satisfaction resulting from the enthusiastic discharge of duty, and to many of us it is the only reward.

Success has been so universal that the public has grown to regard engineering accomplishment as a matter of course. Failure means oblivion. Many of us find ourselves in the position similar to that of a small boy whose duty it was to work the garden. A stranger passing one day saw him leaning disconsolately on his hoe and gazing into the beyond. The stranger said, "Son, do you work here?" "Yes, sir," was the reply. "What pay do you get for it?" "Nothing if I do and a licking if I don't."

Fortunately that is not the situation of all of us. To one in seven thousand members once each year comes the highest honor which can be conferred upon an engineer. Though there may be many who are worthy, the natural limitations are such that only a few can attain it.

It is not given to us to know whether our being in at the death is due to our staying qualities or to a fortunate short cut.

However this may be, matters not to the one who is chosen. I value the honor more than any possible commercial reward. I trust your confidence has not been misplaced, and promise to give my best efforts toward the welfare of this Society, watchful always of its dignity and respecting its traditions. I thank you. (Great applause.)

The Secretary.—Mr. President, so far as the Secretary is concerned, there is no further business. I would suggest, however, sir, that announcement be made, or I can make it, that a meeting of the Board of Direction will be held in the Secretary's office, in the Board Room, rather, at the front entrance of the house, at 1 o'clock, and that all members of the Board are asked to be there promptly, so that the new Board may organize.

Mr. President, for several years, three I think, the luncheon which was served in this house was discontinued, due to the fact that we had such a large attendance when the luncheon was announced that we could not very well serve the people that came. At every Annual Meeting that luncheon has been asked for vociferously, and this year the Committee decided to do the best it could and so a very simple luncheon has been provided, consisting largely of sandwiches and lemonade, and it is hoped that everybody will get something at least, and the Board probably can eat enough before one o'clock to satisfy them.

The President.—My first official action is to declare the meeting adjourned. **Adjourned.**

EXCURSIONS AND ENTERTAINMENTS AT THE SIXTY-FIRST ANNUAL MEETING

Wednesday, January 21st, 1914.—After the Business Meeting, lunch for about 770 members was served at 1 P. M. at the Society House.

At 2.30 P. M., through the courtesy of the Public Service Commission for the First District, Alfred Craven, M. Am. Soc. C. E., Chief Engineer, and the contractors for the various sections of the new subways below Prince Street, Manhattan, about 300 members, made an inspection trip through several portions of the Broadway subway and the tunnels under St. Paul's Churchyard and Vestry Building.

At 9 P. M. there was a reception, with dancing, at the Hotel Astor, at which the attendance was 740.

Thursday, January 22d, 1914.—The day was devoted to an excursion to the Martin's Creek and Tunkhannock Viaducts, now in course of construction by the Delaware, Lackawanna and Western Railroad, George J. Ray, M. Am. Soc. C. E., Chief Engineer. The party of about 450 assembled at Hoboken, N. J., at 8.15 A. M., and the trip was made on a special train. Lunch was served on the train, through the courtesy of the contractors on the bridge work.

The new viaducts are on the new change of line which the Lackawanna Railroad Company is building between Clark Summit and Hallstead, Pa., which line, when completed, will reduce materially the operating costs between these points. The excursion also afforded an excellent opportunity to inspect the long New Jersey Cut-off previously constructed by this company.

The special train was provided with a wireless telegraph outfit and was in communication with New York and Scranton.

The train left Nicholson, Pa., at about 2.30 P. M., and arrived in Hoboken at 6.30 P. M.

In the evening, at the Society House, there was a social and informal "Smoker," at which the attendance was about 800.

The following list contains the names of 999 members of various grades who registered as being in attendance at the Annual Meeting. The list is incomplete, as some members failed to register, and it does not contain the names of any of the guests of the Society or of individual members. The number of guests is estimated at about 400.

Abbott, C. P. . White Plains, N. Y.
Adams, E. E. . . . New York City
Aiken, W. A. . . . Philadelphia, Pa.
Aims, W. I. . . . . . . New York City
Alexander, H. J., White Plains, N. Y.
Allaire, D. A. . . . Brooklyn, N. Y.
Allardice, E. R. B. . . Clinton, Mass.
Allen, F. W. . White Plains, N. Y.
Allen, Kenneth. . . . New York City
Alison, T. H. . . . . . Bayonne, N. J.
Allison, J. E. . . . . . . St. Louis, Mo.
Alsberg, Julius. . . . New York City
Ammann, O. H. . . . New York City
Appleton, T. A. . . . . Beverly, Mass.
Armstrong, R. S. . . New York City

Armstrong, R. W. .New York City
Arnold, W. H. . . . . .New York City
Ashbaugh, L. E. . . . .New York City
Atkinson, A. .New Brunswick, N. J.
Atwood, T. C. . .New Haven, Conn.
Atwood, W. G. . .Indianapolis, Ind.
Auryansen, F. . . . . . .Jamaica, N. Y.

Babcock, W. S. . . . .New York City
Backes, W. J. .New Haven, Conn.
Baird, H. C. . . . . . .New York City
Baker, E. B. . . . . . .Herkimer, N. Y.
Bamford, W. B. . . . .Belmar, N. J.
Banks, C. W. .Pleasantville, N. Y.
Barbour, F. A. . . . . . . .Boston, Mass.
Barker, C. W. T. .Philadelphia, Pa.
Barnes, M. G. . . . . . . .Albany, N. Y.
Barnes, T. H. . . . . . .New York City
Barnett, R. P. . . . . .New York City
Barney, P. C. . . . . .Brooklyn, N. Y.
Barney, W. J. . . . . . .New York City
Basinger, J. G. . . .New York City
Beach, W. N. . . . . . .New York City
Bean, G. L. . . . . . .Philadelphia, Pa.
Beaty, R. E. . . . . . .New York City
Becker, R. C. . . . . . .New York City
Beebe, J. C. . . . . .Havre, Mont.
Beekman, J. V., Jr. .Boston, Mass.
Belcher, W. E. . . . .New York City
Belknap, F. W. . . . .New York City
Bellows, S. R.,
West New Brighton, N. Y.
Belzner, Theodore. .New York City
Benedict, F. N. .Mt. Vernon, N. Y.
Bentley, J. C. . . . . . .Elizabeth, N. J.
Berger, Bernt. . . . . .New York City
Berger, John. . . . . . .New York City
Berry, J. B. . . . . .Oak Park, Ill.
Bettes, C. R. .Far Rockaway, N. Y.
Betts, F. K. . . . . . .Kingston, N. Y.
Betts, R. T. . . . . .New York City
Beugler, E. J. . . . . .New York City
Bevan, L. J. . . . . . . .New York City
Bilyeu, C. S. . . . . . .New York City
Bingham, C. A. . . . .Elizabeth, N. J.

Black, E. F. . . . . . .New York City
Blackmore, G. G. .New York City
Blair, Alexander. . . .Summit, N. J.
Blair, C. M. . . .New Haven, Conn.
Blakeslee, C. . . .New Haven, Conn.
Blakeslee, H. L. .New Haven, Conn.
Blanchard, A. H. .New York City
Blossom, Francis. . .New York City
Boardman, C. S. . . .Buffalo, N. Y.
Boardman, W. H. . .Newark, N. J.
Boes, F. C.,
West New Brighton, N. Y.
Bogart, John. . . . . .New York City
Bogert, C. L. . . . . . .New York City
Boller, A. P., Jr.,
East Orange, N. J.
Boniface, A. . . . . . .Scarsdale, N. Y.
Booth, G. W. . . . . . .New York City
Booz, H. C. . . . . . .Philadelphia, Pa.
Boucher, W. J. . . .New York City
Boughton, W. H.,
Poughkeepsie, N. Y.
Bowditch, J. H.,
New Brighton, N. Y.
Bowman, A. L. . . .New York City
Bowman, D. W. .Phœnixville, Pa.
Boyd, J. C. . . . . . .Montclair, N. J.
Boyd, R. W. . . . . . .New York City
Brace, J. H. .Cedars, Que., Canada
Brackenridge, J. C.,
Richmond Hill, N. Y.
Brackett, Dexter. . . .Boston, Mass.
Bradbury, R. R.,
Pleasantville, N. Y.
Bradley, F. E. . . . .New York City
Brainerd, H. A. . .Westfield, N. J.
Bramwell, G. W. . .New York City
Branne, J. S. . . .Mt. Vernon, N. Y.
Braunworth, P. L. .Roseland, N. J.
Breithaupt, W. H.,
Berlin, Ont., Canada
Breitzke, C. F. . . . . .Boonton, N. J.
Brennan, J. L. . . .New York City
Breuchaud, Jules. .New York City
Briggs, W. C. . . . . .Brooklyn, N. Y.

Brink, L. C.......New York City
Brodie, O. L.,
West New Brighton, N. Y.
Brooks, D. W....New York City
Brooks, J. P......Potsdam, N. Y.
Brown, A. T..White Plains, N. Y.
Brown, C. E.,
Hudson Heights, N. J.
Brown, D. H......Newark, N. J.
Brown, E. H....Hempstead, N. Y.
Brown, G. C.........Ithaca, N. Y.
Brown, L. L......New York City
Brown, S. P.,
Montreal, Que., Canada
Bruce, J. M......New York City
Brumley, D. J........Chicago, Ill.
Brunner, John.......Chicago, Ill.
Brush, W. W......New York City
Bryan, C. W......New York City
Buck, H. R.......Hartford, Conn.
Buel, A. W........New York City
Buel, E. D........New York City
Buettner, O. G. H..New York City
Burden, James....Oswego, N. Y.
Burdett, F. A......New York City
Burgess, G. H......Albany, N. Y.
Burnham, F. W....New York City
Burpee, G. W......New York City
Burr, W. H.......New York City
Burroughs, H. R..New York City
Burrowes, H. G..Mt. Vernon, N. Y.
Burrowes, R. W.,
Long Beach, N. Y.
Bush, E. W..........Lyme, Conn.
Bush, H. D........Baltimore, Md.
Bush, Lincoln..East Orange, N. J.
Bushnell, H. E...Bloomfield, N. J.

Cahn, Elias.......New York City
Campbell, C. C..Philadelphia, Pa.
Cantwell, H. H....Yonkers, N. Y.
Carey, E. G..White Plains, N. Y.
Carle, N. A........Newark, N. J.
Carmalt, L. J....Hartford, Conn.
Carpenter, A. W..Yonkers, N. Y.
Carr, Albert..East Orange, N. J.
Carter, E. C.........Chicago, Ill.
Carter, F. H....Cambridge, Mass.
Casani, A. A......New York City
Casler, M. D..Mt. Vernon, N. Y.
Chase, C. F..New Britain, Conn.
Cheney, H. N...Dorchester, Mass.
Chester, J. N....Pittsburgh, Pa.
Chevalier, L....Jersey City, N. J.
Choate, J. K......New York City
Christian, G. L....New York City
Christie, W. W....Paterson, N. J.
Christy, G. L....New York City
Churchill, C. S.....Roanoke, Va.
Claflin, W. B.,
West Redding, Conn.
Clapp, S. K......Ashokan, N. Y.
Clark, A. E......New York City
Clark, E. W..Pleasantville, N. Y.
Clark, G. H......New York City
Clarke, G. C......New York City
Clarke, St. J......Bogota, N. J.
Cleveland, L. B..Watertown, N. Y.
Codwise, H. R....Brooklyn, N. Y.
Coe, Robert......Pittsburgh, Pa.
Cogswell, W. B...Syracuse, N. Y.
Cohen, A. B......Hoboken, N. J.
Cohen, F. W.,
Upper Montclair, N. J.
Cole, E. S........New York City
Cole, G. N........New York City
Coleman, J. F..New Orleans, La.
Coltman, R., Jr...Elmhurst, N. Y.
Comber, S. X.....New York City
Conard, W. R..Burlington, N. J.
Condron, T. L......Chicago, Ill.
Conger, A. A.,
Shelburne Falls, Mass.
Connell, H. L....New York City
Connell, W. H..Philadelphia, Pa.
Connelly, J. A. A..New York City
Cook, F. S........Yonkers, N. Y.
Cook, J. H........Passaic, N. J.
Coombs, A. W....New York City
Coombs, R. D....New York City

Cooper, D. R.....New York City
Cornell, J. N. H..New York City
Corthell, A. B..Winchester, Mass.
Crane, A. S......New York City
Crane, J. S........Newark, N. J.
Craven, Alfred....Yonkers, N. Y.
Creager, W. P..Ridgewood, N. J.
Crehore, W. W....New York City
Cresson, B. F., Jr..New York City
Crooks, C. H....New York City
Crosby, W. W....Baltimore, Md.
Crowell, Foster...New York City
Cuddeback, A. W..Paterson, N. J.
Culyer, T. C.
Purdy Station, N. Y.
Cummin, Hart....New York City
Cummings, Noah..New York City
Cummings, R. A..Pittsburgh, Pa.
Cuntz, W. C.....New York City
Currier, C. G....New York City
Curtis, C. E......Johnstown, Pa.
Curtis, F. S........Boston, Mass.
Curtis, V. P....Worcester, Mass.
Cushing, W. C....Pittsburgh, Pa.
Cutting, G. W., Jr..Boston, Mass.

Dahl, S. T......New York City
Dailey, J. A..East Orange, N. J.
Dakin, A. H., Jr..New York City
Dana, R. T......New York City
Darling, J. H....Duluth, Minn.
Darrach, C. G..Philadelphia, Pa.
Davies, J. V......New York City
Davis, A. P....Washington, D. C.
Davis, B. H......New York City
Davis, C. E.....Philadelphia, Pa.
Davis, J. L....Mt. Vernon, N. Y.
Day, E. B.......New York City
Dean, A. W........Boston, Mass.
Deans, J. S.....Phœnixville, Pa.
DeGraff, H. W..Amsterdam, N. Y.
Deiser, N. A....Brooklyn, N. Y.
Develin, R. G..Philadelphia, Pa.
Devin, George..Kansas City, Mo.
DeWitt, P. H..Phillipsburg, N. J.
Deyo, S. L. F....New York City
Dibert, H. M........Troy, N. Y.
Diebitsch, E........Nutley, N. J.
Diehl, G. C........Buffalo, N. Y.
Dilks, L. C......New York City
Dixon, G. G..........Kent, Ohio
Dodge, S. D......Cornwall, N. Y.
Donham, B. C..Glen Ridge, N. J.
Donle, E. R......New York City
Doron, C. S......Brooklyn, N. Y.
Dorrance, W. T....Boston, Mass.
Douglas, W. J....New York City
Douty, D. E......New York City
Downman, J. R.,
Washington, D. C.
Doyen, G. E......New York City
Dufour, F. O........Athens, Pa.
Durfee, J. J......New York City
Durham, H. W....New York City
Durham, L......New Paltz, N. Y.
Dykeman, C. F..Brooklyn, N. Y.

Earle, Thomas......Steelton, Pa.
Easby, M. W....Philadelphia, Pa.
Easterbrook, F. J.,
New Haven, Conn.
Eckersley, J. O..New York City
Eddy, H. P........Boston, Mass.
Edmondson, R. S..New York City
Edwards, D. G..Brooklyn, N. Y.
Edwards, J. H.....Passaic, N. J.
Edwards, W. R....Baltimore, Md.
Ehle, Boyd, Victoria, B. C., Canada
Ehrbar, L. H......New York City
Eide, Torris.......New York City
Elliott, C. G....Washington, D. C.
Elliott, J. W.....Burlington, Vt.
Ellis, G. W.......Hornell, N. Y.
Ellis, H. C....White Plains, N. Y.
Ellis, J. W....Woonsocket, R. I.
Ely, C. B........Harrisburg, Pa.
Emerson, K. B....New York City
Endemann, H. K.,
Long Island City, N. Y.
Endicott, M. T..Washington, D. C.

Entenmann, P. M..Brooklyn, N. Y.
Ewing, W. W....Westfield, N. J.

Farley, J. M. .White Plains, N. Y.
Farley, M. M......New York City
Farnham, A. B...Pittsfield, Mass.
Farnham, R., Jr. .Philadelphia, Pa.
Farrington, H. P. .New York City
Federlein, W. G. .New York City
Ferguson, L. R. .Philadelphia, Pa.
Fernstrom, H.......Norfolk, Va.
Ferry, C. A...New Haven, Conn.
Fetherston, J. T...New York City
Files, T. H.......New York City
Firth, E. W......Jamaica, N. Y.
Fisher, E. A.....Rochester, N. Y.
FitzGerald, D....Brookline, Mass.
Flinn, A. D......Yonkers, N. Y.
Floy, Henry........New York City
Flynn, G. A......New York City
Forbes, F. B......New York City
Ford, H. C........New York City
Forrest, C. N......Maurer, N. J.
Fortin, S. J., Montreal, Que., Canada
Foss, F. E........New York City
Fougner, H.......New York City
Francis, W. J., Montreal, Que., Canada
Franklin, C. M....New York City
Frazee, J. H......New York City
French, A. H....Brookline, Mass.
French, A. W...Worcester, Mass.
French, C. R...Wilkes-Barre, Pa.
French, Halsey. .Brooklyn, N. Y.
French, J. B......New York City
Frost, G. S......Brooklyn, N. Y.
Fuller, G. W......New York City
Fuller, W. B......New York City
Fuller, W. E......New York City
Furber, W. C...Philadelphia, Pa.

Gahagan, W. H...Brooklyn, N. Y.
Gandolfo, J. H....New York City
Gardiner, F. W....Yonkers, N. Y.
Gardner, Warren. ..New York City
Garfield, C. A..White Plains, N. Y.
Gartensteig, C....New York City
George, W. W., East Liverpool, Ohio
Gerber, Emil......Pittsburgh, Pa.
Gerhard, N. P. .Scarsdale, N. Y.
Gibboney, F. L......Roanoke, Va.
Gibbs, E. A......Pittsburgh, Pa.
Giesey, J. K......New York City
Gifford, G. E......New York City
Gildersleeve, A. C. .New York City
Giles, Robert......New York City
Gillespie, R. H....New York City
Gilman, Charles. .Plainfield, N. J.
Gladding H. H..New Haven, Conn.
Goldsborough, J. B.. .Croton, N. Y.
Goldsmith, W....New York City
Goodell, J. M., Upper Montclair, N. J.
Goodman, Joseph. .New York City
Goodman, Louis....New York City
Goodrich, E. P.. .Brooklyn, N. Y.
Goodsell, D. B....New York City
Goodwin, R. E....New York City
Gould, C. M. .Cold Spring, N. Y.
Gould, W. T....Hastings, N. Y.
Gowen, J. F.....Ossining, N. Y.
Graham, G. A....New York City
Granbery, J. H....New York City
Grantham, H. T..Philadelphia, Pa.
Gravell, W. H. .Philadelphia, Pa.
Gray, William.....New York City
Greathead, J. E....New York City
Green, C. N.......New York City
Greene, C.........New York City
Greene, G. S., Jr. .New York City
Greenlaw, R. W....New York City
Gregg, J. H. C. .Brooklyn, N. Y.
Gregory, C. E....Mt. Kisco, N. Y.
Greiner, J. E.....Baltimore, Md.
Griffith, W. F. R.....Norfolk, Va.
Gudmundsson, G. .Pittsburgh, Pa.
Guise, Philip.....Brooklyn, N. Y.
Guthrie, K. O..Schenectady, N. Y.

Guttridge, J. A.,
Brown Station, N. Y.

Haas, P. L.......New York City
Haines, E. G....Brooklyn, N. Y.
Hale, H. M......Brooklyn, N. Y.
Hall, M. W........New York City
Hall, W. M..Parkersburg, W. Va.
Hallihan, J. P....New York City
Hallock, J. C......Newark, N. J.
Halsey, W. H.,
Bridgehampton, N. Y.
Hamilton, E. P....New York City
Hamilton, J. W...New York City
Hammond, G. T..Brooklyn, N. Y.
Hammond, J. F.,
Richmond Hill, N. Y.
Hammond, J. M..Kansas City, Mo.
Hanavan, W. L....New York City
Hansel, Charles..Cranford, N. J.
Harding, H. S.....New York City
Haring, Alex......New York City
Harps, H. M....Brooklyn, N. Y.
Harrington, J. L.,
Kansas City, Mo.
Harris, F. R....Brooklyn, N. Y.
Harris, Stephen.Philadelphia, Pa.
Harrison, L. B....New York City
Harte, C. R....New Haven, Conn.
Hartman, A. F....Nutley, N. J.
Harwi, S. J......Bayonne, N. J.
Haskell, E. E......Ithaca, N. Y.
Haskins, W. J....New York City
Hatt, W. K......Lafayette, Ind.
Hatton, H. W..Wilmington, Del.
Hatton, T. C....Wilmington, Del.
Hauck, William...New York City
Hazard, E......Wilkinsburg, Pa.
Hazelton, C. W.,
Montague City, Mass.
Hazen, Allen......New York City
Hazen, W. N..East Orange, N. J.
Heald, E. C...Washington, D. C.
Healy, J. R.......New York City
Hedges, S. H......Seattle, Wash.
Heilbronner, L. C....Utica, N. Y.
Heiser, A. B....Brooklyn, N. Y.
Heller, J. W......Newark, N. J.
Henderson, J. T..Hartford, Conn.
Henry, P. W......New York City
Hering, Rudolph..Montclair, N. J.
Hermanns, F. E...New York City
Hewes, V. H......New York City
Higgins, C. H..Jersey City, N. J.
Higgins, J. W..Roselle Park, N. J.
Higginson, J. Y...New York City
Hill, W. R........Albany, N. Y.
Hilton, J. C....Brooklyn, N. Y.
Hilts, H. E.....Philadelphia, Pa.
Himes, A. J......Cleveland, Ohio
Hocke, J. G......Bayonne, N. J.
Hodgdon, B. A....New York City
Hodgdon, F. W....Boston, Mass.
Hodge, H. W......New York City
Hogan, J. P.......New York City
Holbrook, A. R..Brooklyn, N. Y.
Holbrook, P.......New York City
Holden, C. A..Mt. Vernon, N. Y.
Holdredge, N. C...New York City
Holland, C. M....Brooklyn, N. Y.
Holmes, N. H........Chicago, Ill.
Holtzman, S. F..Hastings, N. Y.
Honness, G. G....Kingston, N. Y.
Hood, J. N.,
Quebec, Que., Canada
Horne, H. W..Farmington, Conn.
Hortenstine, R......Dallas, Tex.
Horton, R. E......Albany, N. Y.
Hovey, O. E......Plainfield, N. J.
Howard, L. T.....New York City
Howard, O. Z....New York City
Howe, C. E......Hastings, N. Y.
Howe, W. T..West Medford, Mass.
Howell, D. J..Washington, D. C.
Howes, D. W....Brooklyn, N. Y.
Hoyt, J. C....Washington, D. C.
Hubbard, W. D.,
Brown Station, N. Y.
Hudson, C. W.,
Upper Montclair, N. J.

Hudson, D. S. . . . Astoria, N. Y.
Hudson, H. W. . . . . New York City
Hughes, H. J. . Cambridge, Mass.
Hulburd, L. S. . Seneca Falls, N. Y.
Hulsart, C. R. . . . . New York City
Humphrey, R. L., Philadelphia, Pa.
Humphreys, A. C. . New York City
Hunt, C. A. . . . . New York City
Hunt, C. E. . . . . New York City
Hunt, Chas. Warren, New York City
Hunt, R. W. . . . . Chicago, Ill.
Hunter, R. E. . . . Yonkers, N. Y.
Huntington, L. M., New York City
Hurd, H. L. . White Plains, N. Y.
Hurley, J. P. . . . Brooklyn, N. Y.
Hutchins, H. C. . . . New York City
Hyde, J. L. . . . . . Westfield, Mass.

Immediato, G. . . . Montclair, N. J.
Ingersoll, C. M. . . . New York City
Irwin, J. C. . . . . Boston, Mass.

Jackson, D. C. . Brookline, Mass.
Jacobs, R. H. . . . . . New York City
Jacobsen, H. P. R.. Brooklyn, N. Y.
Janes, G. P. . . . . Roselle, N. J.
Janni, A. C. . . . . . . New York City
Jarrett, E. S. . . . . . New York City
Jenkins, J. E. . . . . . New York City
Johannesson, S., Upper Montclair, N. J.
Johnston, J. A. . . Springfield, Mass.
Jones, J. . . . . Philadelphia, Pa.
Jones, S. R. . . . . . . New York City
Jones, W. A. . New Haven, Conn.
Jonson, E. F. . . . . New York City
Judd, W. M. . . . . . Pittsburgh, Pa.
Just, G. A., Long Island City, N. Y.

Karner, W. J. . . . . . New York City
Kayser, E. M. . . . Lock Raven, Md.
Keays, R. H. . . . New Paltz, N. Y.
Keefer, C. H., Ottawa, Ont., Canada
Keith, H. C. . . . . . . New York City
Kelley, W. D. . . . . . Yonkers, N. Y.
Kellogg, R. C. . . . Brooklyn, N. Y.
Kelly, C. W. . . . New Haven, Conn.
Kennedy, P. J. . . . Holyoke, Mass.
Khuen, Richard. . Pittsburgh, Pa.
Kiehm, Charles. . . . . . Utica, N. Y.
Kimball, F. C. . . . Summit, N. J.
King, E. T. . . . . . Arrochar, N. Y.
King, P. S. . . . Wilmington, Del.
King, W., Jr. . . . . . New York City
Kinney, W. M. . . . Pittsburgh, Pa.
Kinsey, W. A. . . . . . Newark, N. J.
Kinsley, T. P. . . . . . New York City
Kirkwood, H. C. . Flushing, N. Y.
Kittredge, G. W. . Yonkers, N. Y.
Knap, E. D. . . . Tuckahoe, N. Y.
Knickerbocker, C. E., New York City
Knighton, J. A. . . . New York City
Knowles, Morris. . Pittsburgh, Pa.
Knox, S. B. . . . . New York City
Kolb, H. J. . . . . New York City
Kornfeld, A. E. . . . New York City
Kraus, Arthur. . . . . New York City
Krellwitz, D. W. . . New York City
Kuchar, F. M. . . . . . New York City
Kuichling, Emil. . . New York City

La Chicotte, H. A., Brooklyn, N. Y.
Lamson, W. M. . . . New York City
Landers, C. S. . . . . New York City
Landreth, O. H., Schenectady, N. Y.
Lang, F. A. . . . . New York City
Lange, T. F. . . . . . . New York City
Langthorn, J. S. . . New York City
Lannan, L. E. . . . . . New York City
Larsson, C. G. E. . New York City
Lathrop, J. C. . . . . Baltimore, Md.
Latimer, C. A. . Beechhurst, N. Y.
Latta, H. W. . . . Philadelphia, Pa.

Lavis, F. . . . . .New York City
Lawrence, E. V. . . .New York City
Lee, E. H. . . . . .Chicago, Ill.
Lee, W. B. . . . . .New York City
Leeuw, H. A. . . . . .New York City
Leighton, G. . . . . . .Glenburn, Pa.
Lenderink, A. . .Kalamazoo, Mich.
Leonard, H. R. .Philadelphia, Pa.
Leser, H. . . . . .New York City
Lesley, R. W. . . .Philadelphia, Pa.
Lester, W. J. . . . . .New York City
Lewis, E. C. . . . . .Nashville, Tenn.
Lewis, E. W. . . .New Haven, Conn.
Lewis, J. O. . . . . .Brooklyn, N. Y.
Lewis, L. H. . . . . . .New York City
Lewis, N. P. . . . . . .New York City
Lex, W. I. . . . . .Philadelphia, Pa.
Lichtenstein, H. . . .New York City
Lichtner, W. O., Newton Highlands, Mass.
Lieb, J. W., Jr. . . .New York City
Llewellyn, F. T. . . .New York City
Lober, J. B. . . . .Philadelphia, Pa.
Lobo, Carlos. . . .Brooklyn, N. Y.
Lockwood, W. D., Rockville Center, N. Y.
Loewenstein, J. . . .New York City
Logan, W. S. . . . . .Newark, N. J.
Longley, F. F. . . . . .New York City
Look, M. J. .Brown Station, N. Y.
Loomis, Horace, Mt. Vernon, N. Y.
Loughran, J. F. .Kingston, N. Y.
Low, G. E. . . .Maplewood, N. J.
Loweth, C. F. . . . . . .Chicago, Ill.
Lowinson, O. . . . . . .New York City
Lucas, G. L. . . . . . .New York City
Lucius, Albert. . . .Brooklyn, N. Y.
Lueder, A. B. . . . . .New York City
Lum, D. W. . . .Washington, D. C.
Lundie, John. . . . . .New York City
Lyons, H. C. . . . . . .Flushing, N. Y.

MacCornack, C. W., Phœnixville, Pa.
MacDonald, C. . . .Trenton, N. J.
MacFeeters, J. O., Bloomfield, N. J.
MacGregor, R. A. .New York City
Machen, H. B. . . . .New York City
MacIlvaine, F. S. .Trenton, N. J.
Mackenzie, K. G. .Bayonne, N. J.
McCarthy, D. F. .Bronxville, N. Y.
McCarty, R. J. .Kansas City, Mo.
McClave, S. W., Jr., Cliffside, N. J.
McClintock, J. R. .New York City
McComb, D. E. .Washington, D. C.
McConnell, I. W. . . .Boston, Mass.
McDonald, Hunter, Nashville, Tenn.
McInnes, F. A. . . .Boston, Mass.
McIntyre, W. A. .Philadelphia, Pa.
McKenzie, T. H., Southington, Conn.
McLure, N. R. . . .Phœnixville, Pa.
McMinn, T. J. . . .Brooklyn, N. Y.
McMullen, R. W., New Canaan, Conn.
McNab, William, Montreal, Que., Canada
McNaugher, D. W., Pittsburgh, Pa.
McPherson, R. H. .New York City
Madden, J. H. . . . .New York City
Maltby, F. B., Mechanicsville, N. Y.
Manley, L. B., West Roxbury, Mass.
Mann, J. L., Santo Domingo, Santo Domingo
Mansfield, W. H. . . . .Troy, N. Y.
Marburg, Edgar. .Philadelphia, Pa.
Marple, W. M. . . . .Scranton, Pa.
Marsh, F. B. . . . . . .New York City
Martin, E. D. . . . . .New York City
Martin, John. . . . . .Albany, N. Y.
Mason, Francis. . . .New York City
Matheson, J. D., Montreal, Que., Canada

Matlaw, I. S. ....New York City
Mayo, G. W. .....Smethport, Pa.
Mazeau, C. .....Brooklyn, N. Y.
Mead, C. A.,
Upper Montclair, N. J.
Meadowcroft, W. ..New York City
Meem, J. C. ......New York City
Mehren, E. J. .....New York City
Melius, L. L. .....New York City
Menke, William. ..New York City
Mercer, C. H. ....Steelton, Pa.
Merrill, Ogden. ...New York City
Merriman, Thaddeus,
New York City
Miles, G. F. ....New York City
Miller, A. B. ......New York City
Miller, M. M. .....Yonkers, N. Y.
Miller, R. P. ......New York City
Mills, A. L. ....Toledo, Ohio
Mitchell, S. P. .Philadelphia, Pa.
Mogensen, O. E. ..New York City
Moisseiff, L. S. ...New York City
Molitor, D. A.,
Toronto, Ont., Canada
Molitor, F. ....New York City
Monsarrat, C. N.,
Montreal, Que., Canada
Montfort, R. .....Louisville, Ky.
Moore, C. H. .....New York City
Moore, F. F. ...Hawthorne, N. Y.
Moore, S. W.,
East Elmhurst, N. Y.
Moore, W. H. .New Haven, Conn.
Morrison, H. J. .Prospect, N. Y.
Morrison, R. L. .Philadelphia, Pa.
Morse, C. F. ....Malden, Mass.
Morton, W. S. ....New York City
Moses, J. C. ...Cambridge, Mass.
Moss, W. B. .Tompkinsville, N. Y.
Muirhead, J. H. H.,
London, England
Munkelt, F. H. ...Brooklyn, N. Y.
Murphy, J. F.,
Tompkinsville, N. Y.
Murphy, J. J. .....Detroit, Mich.
Musson, E. F. ...Norwich, N. Y.
Myers, C. H. ......New York City
Myers, J. H. .White Plains, N. Y.

Neely, W. R. ...Tarrytown, N. Y.
Neff, F. H. .....Cleveland, Ohio
Nelson, F. B. .....New York City
Nelson, J. W. .....New York City
Neumeyer, R. E. ..Bethlehem, Pa.
Newell, F. H. .Washington, D. C.
Newton, A. W. ......Chicago, Ill.
Nichols, C. H.,
New Haven, Conn.
Noble, Alfred. ....New York City
Northam, M. P. ...New York City
Norton, A. G. .Middletown, N. Y.
Nugent, P. C. ....Syracuse, N. Y.

Obreiter, J. W. .Jersey City, N. J.
O'Brien, J. H.,
Montreal, Que., Canada
O'Connor, J. A. ...Albany, N. Y.
Ockerson, J. A. ...St. Louis, Mo.
Oestreich, H. L. .Brooklyn, N. Y.
Ogden, J. C. ...Plainfield, N. J.
Olmsted, A. E. ....New York City
O'Rourke, J. F. ...New York City
Osbourn, H. V. B.,
Germantown, Pa.
Osgood, J. O. .....New York City
Ott, S. J. .....Rutherford, N. J.
Overocker, D. W. .Syracuse, N. Y.
Owen, James. ...Montclair, N. J.
Oxholm, T. S.,
West New Brighton, N. Y.

Paddock, H. C. ...New York City
Page, L. W. .Washington, D. C.
Pagon, W. W. ...Baltimore, Md.
Palmer, G. B. .....New York City
Palmer, S. B. ...Norwich, Conn.
Parker, C. J. ...Bronxville, N. Y.
Parker, J. L. .....New York City
Parmley, W. C.,
Upper Montclair, N. J.

Parry, F. H........Kingston, Pa.
Parsons, G. W......Steelton, Pa.
Parsons, H. de B. .New York City
Peabody, W. W.,
White Plains, N. Y.
Pegram, G. H.....New York City
Pellissier, G. E.,
Springfield, Mass.
Pelz, C. E.........New York City
Pemoff, J. J.......New York City
Perkins, P. S....Providence, R. I.
Pfau, J. W.......New York City
Philips, J. H. .Glen Ridge, N. J.
Phillips, A. E. .Washington, D. C.
Phillips, H. C......Chicago, Ill.
Pierce, F. E.......New York City
Pinner, Guy.......New York City
Pitcher, S. H....Worcester, Mass.
Pohl, C. A........New York City
Pommerer, R. W.,
Mt. Vernon, N. Y.
Pond, H. O.......Tenafly, N. J.
Porter, H. G......Yonkers, N. Y.
Porter, J. M.........Easton, Pa.
Post, H. W......Boonton, N. J.
Potter, A..........New York City
Potter, H. L....Brooklyn, N. Y.
Potts, Clyde.......New York City
Powers, C. V. V. . New York City
Pratt, A. H. .White Plains, N. Y.
Pratt, F. E....Glen Ridge, N. J.
Preston, G. H. .Bloomfield, N. J.
Preston, H. W....Elmira, N. Y.
Price, C. P........Malden, Mass.
Price, F. O.......Brooklyn, N. Y.
Price, P. L........Bayside, N. Y.
Prichard, H. S....Pittsburgh, Pa.
Priest, B. B......New York City
Proctor, R. F....Baltimore, Md.
Pulligny, J. L. de. .New York City
Purdy, S. M....Brooklyn, N. Y.
Purver, G. M.....Brooklyn, N. Y.

Quimby, C. H., Jr.,
Mt. Vernon, N. Y.
Quimby, H. H.,
Philadelphia, Pa.
Quincy, C. F......New York City
Quinn, M. F......New York City

Raff, H. G........New York City
Rankin, E. S....Newark, N. J.
Ray, G. J....East Orange, N. J.
Reeves, W. F......New York City
Reimer, A. A., East Orange, N. J.
Reimer, F. A., East Orange, N. J.
Reimer, W. H. V.,
East Orange, N. J.
Rennell, H. H.....New York City
Reynders, J. V. W.,
Steelton, Pa.
Rhett, A. H......Brooklyn, N. Y.
Rice, G. S........New York City
Richardson, C.....New York City
Richardson, J. H. . .Boston, Mass.
Richardson, T. F. .Brooklyn, N. Y.
Richmond, J. P. W., Yonkers, N. Y.
Ricker, G. A......Albany, N. Y.
Ridgway, Robert. . .New York City
Riggs, T., Jr. .Washington, D. C.
Rights, L. D......New York City
Ripley, B.,
Toronto, Ont., Canada
Ripley, H. L.....Brockton, Mass.
Ripley, Joseph....Albany, N. Y.
Robbins, F. H.....New York City
Roberts, H. W.....New York City
Roberts, R. F......Nutley, N. J.
Robinson, E. F....New York City
Robinson, G. L....New York City
Robinson, R. T. .Mt. Vernon, N. Y.
Rockwood, N. C. .Brooklyn, N. Y.
Rogers, A. W....Rochester, N. Y.
Rogers, E. H. .West Newton, Mass.
Rogers, H. L......New York City
Rogge, J. C. L....New York City
Rossi, Irving......New York City
Rourke, L. K......Roxbury, Mass.
Rugg, W. F......Peekskill, N. Y.
Rumery, R. R.....New York City

Rust, C. H.,
Victoria, B. C., Canada
Rust, H. P........New York City
Ryan, M. H.......New York City
Ryder, E. M. T..Yonkers, N. Y.
Ryon, Henry....Brooklyn, N. Y.

Safford, A. T......Lowell, Mass.
Sanborn, F. B.,
Tufts College, Mass.
Sanborn, J. F.....New York City
Sanborn, M. F.,
Pleasantville, N. Y.
Sando, W. J....Milwaukee, Wis.
Sargent, P. D......Augusta, Me.
Saunders, W. L....New York City
Saville, C..........New York City
Saville, C. M....Hartford, Conn.
Schaeffer, A.......New York City
Schall, F. E.,
South Bethlehem, Pa.
Schmidt, H. H....Brooklyn, N. Y.
Schneider, C. C.,
Philadelphia, Pa.
Schreiber, J. M....Newark, N. J.
Schroeder, F. C....New York City
Seabury, G. T.,
White Plains, N. Y.
Seaman, H. B....New York City
Seaman, W. H..Glen Cove, N. Y.
Searle, L. F....Kingston, N. Y.
Seaver Clifford....New York City
Senior, F. S..Montgomery, N. Y.
Serber, D. C......New York City
Shailer, R. A....Brookline, Mass.
Shaughnessy, C. S.,
Cold Spring, N. Y.
Shaw, F. H.......Lancaster, Pa.
Shaw, G. H......Brooklyn, N. Y.
Sheffield, E. N....New York City
Shellenberger, L. R.,
Bayonne, N. J.
Shenehon, F. C.,
Minneapolis, Minn.
Shepherd, F. C....Boston, Mass.
Sherman, A. L....Mansfield, Mass.
Sherman, H. J....Camden, N. J.
Shertzer, T. B.....New York City
Shoemaker, M. N..Newark, N. J.
Shute, J. S..Richmond Hill, N. Y.
Simpson, G. F....New York City
Sitt, W. T........New York City
Skinner, F. W.,
Tompkinsville, N. Y.
Skinner, J. F....Rochester, N. Y.
Smith, A...........Bayonne, N. J.
Smith, C. W.......New York City
Smith, E. F.......New York City
Smith, H. J......Putnam, Conn.
Smith, J. R........New York City
Smith, J. W.......New York City
Smith, Joseph.....New York City
Smith, M. H......New York City
Smith, R. B....Brooklyn, N. Y.
Smith, W. F....Valhalla, N. Y.
Smith, W. T......New York City
Smoley, C. K......Scranton, Pa.
Smoyer, L. I......New York City
Smulski, Edward....Boston, Mass.
Snow, C. H.......New York City
Snow, J. B..Richmond Hill, N. Y.
Snow, J. P.........Boston, Mass.
Snyder, F. A......New York City
Snyder, G. D......New York City
Soper, G. A.......New York City
Spaulding, F. A..Pittsfield, Mass.
Spaulding, R. E....Suffield, Conn.
Spear, W. E.......New York City
Spelman, J. R.,
Rockville Center, N. Y.
Spencer, C. B.....New York City
Spencer, F. N......New York City
Spencer, Herbert...New York City
Spencer, W. T..Waterbury, Conn.
Sperry, L. N....Old Forge, N. Y.
Splitstone, C. H...New York City
Spofford, C. M......Boston, Mass.
Sprague, E. L., Jr.,
White Plains, N. Y.
Stark, C. W.......New York City

Starkweather, A. K. . Newark, N. J.
Starr, H. H. . . . . . . New York City
Stearns, F. P. . . . . . Boston, Mass.
Stearns, R. H. . . . Hartford, Conn.
Steere, A. E. . . . . . Waterloo, N. Y.
Stehle, F. C. . . . . Towanda, Pa.
Stepath, C. U. . . . . . New York City
Stern, E. W. . . . . . . New York City
Stevens, H. C. . . . . New York City
Stevens, J. F. . . . . . New York City
Stevenson, W. F.,
New Rochelle, N. Y.
Stewart, S. J. . White Plains, N. Y.
Stiles, A. I. . . . Barrios, Guatemala
Stoddard, R. F. . Bridgeton, Conn.
Stow, F. S. . . . . . Providence, R. I.
Stowitts, G. P. . . . Yonkers, N. Y.
Strachan, Joseph,
Brooklyn, N. Y.
Straub, T. A. . . . . Pittsburgh, Pa.
Strawn, T. C. . . . . . New York City
Strickler, F. W.,
Cambridge Springs, Pa.
Strobel, C. L. . . . . Chicago, Ill.
Strouse, W. F. . . . Baltimore, Md.
Stuart, F. L. . . . . . Baltimore, Md.
Stuart, J. T. . . . . Philadelphia, Pa.
Sullivan, J. F. . . . . New York City
Sutton, Frank. . Washington, D. C.
Swain, G. F. . . . . Boston, Mass.
Swensson, Emil. . . Pittsburgh, Pa.
Swensson, O. J. . Yonkers, N. Y.
Swindells, J. S. . Brooklyn, N. Y.
Sykes, George. . . . . . New York City

Taber, G. A. . . . Brooklyn, N. Y.
Taft, J. R. . . . . New York City
Talbot, A. N. . . . . Urbana, Ill.
Talbot, Earle. . . . . . . Utica, N. Y.
Talbot, F. M. . Glen Ridge, N. J.
Tallman, Leroy. . . . Newark, N. J.
Tallman, P. B. . East Orange, N. J.
Tarr, C. W. . . . . . Lawrence, Mass.
Tate, R. L'H. . . . . New York City
Taylor, A. J. . . . Wilmington, Del.
Taylor, C. F. . . . . . . New York City
Taylor, H. W. . . . . . Albany, N. Y.
Taylor, W. G. . . . . . Newark, N. J.
Terry, J. H. . . . Philadelphia, Pa.
Thacher, Edwin. . . . New York City
Thian, P. E. . . . . . St. Paul, Minn.
Thomas, C. D. . . . Brooklyn, N. Y.
Thomes, E. H. . . . Jamaica, N. Y.
Thompson, J. A. . . . New York City
Thompson, S. C. . . . New York City
Thompson, S. E.,
Newton Highlands, Mass.
Thomson, A., Jr. . . Walden, N. Y.
Thomson, S. F. . New Paltz, N. Y.
Thomson, T. K. . . . New York City
Throop, G. H. . Mamaroneck, N. Y.
Thurber, C. D. . . . Brooklyn, N. Y.
Tidd, A. W. . White Plains, N. Y.
Tighe, J. L. . . . . . . Holyoke, Mass.
Tilden, C. J. . . . . . . Baltimore, Md.
Tillson, G. W. . . . Brooklyn, N. Y.
Timberlake, S. M. . Brooklyn, N. Y.
Tompkins, E. de V.,
New York City
Toner, A. C. . . . . . Baltimore, Md.
Tooker, F. W., East Orange, N. J.
Torrance, W. M.,
East Orange, N. J.
Trautwine, J. C., Jr.,
Philadelphia, Pa.
Travell, W. B. . Greeneville, Tenn.
Travers-Ewell, A. . New York City
Triest, W. G. . . . . . New York City
Trimpi, A. L. . East Orange, N. J.
Trow, F. H. . Brown Station, N. Y.
Tucker, L. W. . . . . . New York City
Tull, R. W. . . . . New York City
Tuska, G. R. . . . . . . New York City
Tuttle, A. S. . . . . New York City
Tyler, W. R. . . . . . . New York City

Ulrich, Daniel. . . . Katonah, N. Y.
Underhill, G. G. . . . Newark, N. J.
Underwood, H. W.,
Philadelphia, Pa.

Ungrich, M. J.....New York City

Vail, J. J..........Rahway, N. J.
Vallely, W. P.....New York City
Van Alstyne, H. A., Herkimer, N. Y.
Van Cleve, A. H...New York City
Van Horne, J. G...New York City
Van Horne, J. R..New York City
Vanneman, C. R....Albany, N. Y.
Van Norden, E. M., Brooklyn, N. Y.
Van Winkle, E. B..New York City
Vaughan, L. B....Kingston, N. Y.
Verveer, E. L......New York City
Vincent, J. I......New York City
Vorce, C. B....New Haven, Conn.
Vredenburgh, W., Jr., Woodmere, N. Y.
Vrooman, M...Gloversville, N. Y.

Wachter, C. L.....New York City
Waddell, F. C., West New Brighton, N. Y.
Waddell, M........New York City
Wadhams, J. P., New Haven, Conn.
Wadsworth, J. E...New York City
Wagner, H. E....Chatham, N. J.
Wagner, J. C....Philadelphia, Pa.
Waite, G. B.......New York City
Waldron, S. P......Boston, Mass.
Walker, C. I......New York City
Walker, E. M......Detroit, Mich.
Walker, J. J..Dobbs Ferry, N. Y.
Walker, J. S....Brooklyn, N. Y.
Walker, J. W....Pittsburgh, Pa.
Wallace, J. F......New York City
Ward, C. D........New York City
Ward, E. A........Newark, N. J.
Wardle, E. B......New York City
Warnock, W. H....New York City
Washington, W. de H., New York City
Wason, L. C....Brookline, Mass.
Watkins, F. W., White Plains, N. Y.
Watson, Merrill....New York City
Watters, G. L., South Bethlehem, Pa.
Webster, A. L.....New York City
Webster, G. S..Philadelphia, Pa.
Webster, W. R..Philadelphia, Pa.
Wegmann, E......Yonkers, N. Y.
Wells, C. E..White Plains, N. Y.
Wells, G. E....Naugatuck, Conn.
Wendt, E. F...Washington, D. C.
Wentworth, G. L..Yonkers, N. Y.
Werbin, I. V......New York City
Weston, F. S....Middleboro, Mass.
Wheatcroft, H. B., Jr., Newark, N. J.
Wheeler, H. R.....New York City
Wheeler, R. C.....New York City
Wheelock, DeF. A...Warren, Pa.
Whinery, S..East Orange, N. J.
Whiskeman, J. P..New York City
White, Lazarus.....New York City
White, W. M....Fanwood, N. J.
Whited, Willis....Harrisburg, Pa.
Whitman, K., Jr...New York City
Whitney, F. O......Boston, Mass.
Whitsit, L. A.....New York City
Whitson, A. U....Flushing, N. Y.
Whittemore, J. O..Hoboken, N. J.
Whittemore, W. F..Hoboken, N. J.
Whittier, T. T.....New York City
Wickham, H. R..Rochester, N. Y.
Wiggin, E. W..New Haven, Conn.
Wiggin, T. H..Scarsdale, N. Y.
Wiggins, W. D....Pittsburgh, Pa.
Wilcock, F......Brooklyn, N. Y.
Wild, H. J..........Chester, Pa.
Wilder, C. W....Brooklyn, N. Y.
Wildes, W. G....Rochester, N. Y.
Wiley, W. H......New York City
Wilgus, W. J......New York City
Wilkins, W. G....Pittsburgh, Pa.
Williams, E. G...Brooklyn, N. Y.
Williams, F. M....Goshen, N. Y.

Williams, G. S...Ann Arbor, Mich.
Wilmot, James....New York City
Wilmot, Sydney....New York City
Wilson, C. W. S.,
New Rochelle, N. Y.
Wilson, James...Montchanin, Del.
Wilson, P. H....Philadelphia, Pa.
Wilson, T. L......New York City
Wilson, W. T......New York City
Wilson, W. W........Berwyn, Pa.
Wilson, William....New York City
Winsor, F. E..White Plains, N. Y.
Winsor, G. A......Valhalla, N. Y.
Winsor, H. D......New York City
Witmer, F. P..East Orange, N. J.
Wolfe, F. C......Baltimore, Md.
Wolfe, F. G........Scranton, Pa.
Wölfel, P. L......Pittsburgh, Pa.
Wood, G. P......Peekskill, N. Y.
Worthington, C..New York City
Wright, F. J.....Paterson, N. J.
Wright, J. B......New York City
Wyckoff, C. R..Hartsdale, N. Y.
Wyman, A. M..East Orange, N. J.

Yates, P. K.......New York City
Yates, W. H......Albany, N. Y.
Yereance, W. B....New York City

Zipser, M. E.,
Cornwall-on-Hudson, N. Y.
Zook, M. A......Plainfield, N. J.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

March 4th, 1914.—8.30 P. M.—A regular business meeting will be held, and a paper by John C. Koch, Assoc. M. Am. Soc. C. E., entitled, "An Investigation of Sand-Clay Mixtures for Road Surfacing", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

March 11th, 1914.—A Special Meeting will be held at the Society House on Wednesday, March 11th, 1914, at 8.30 P. M., for the discussion of the subject of the Valuation of Public Utilities.

The Report of the Special Committee on Valuation for the Purpose of Rate-Making, which was formally presented to the Annual Meeting, will be the special subject for discussion. So much interest has been developed that it is believed an opportunity for debate on the questions involved will be taken advantage of by many. Written discussions on this subject will be received and printed in *Proceedings* as soon after their receipt as possible.

March 18th, 1914.—8.30 P. M.—At this meeting a paper by C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie and C. O. Carey, entitled, "Report on a Series of Tests on Concrete Columns Reinforced with a Spiral of Steel", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-sixth Annual Convention of the Society will be held at Baltimore, Md., from June 2d to 5th, 1914, inclusive.

The following Committees to take charge of arrangements have been appointed:

Committee of the Board of Direction:

James H. Edwards, George W. Fuller,
Chas. Warren Hunt.

Local Committee of Arrangements:

Francis Lee Stuart, *Chairman.*

Mendes Cohen, W. Anderson Polk,
W. W. Crosby, Layton F. Smith,
J. E. Greiner, H. A. Warren,
F. H. Hambleton, E. B. Whitman.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be re-

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

quested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Visiting members are urged to attend the meetings.

(Abstract of Minutes of Meeting)

January 9th, 1914.—The meeting was called to order; President Ridgway in the chair; Roger W. Toll, Secretary; and present, also, 22 members and guests.

The minutes of the meeting of November 14th, 1913, were read and approved. The meeting scheduled for December 19th, 1913, was not held on account of weather conditions.

The President stated that the weekly luncheons would be discontinued until further notice, on account of the reorganization of the Colorado Traffic Club.

The adoption of an emblem by the Association was discussed, and the Secretary was ordered to report on the matter at the next meeting.

The Report of the Special Committee on Valuation of Public Utilities of the Society was discussed by Messrs. Leonard Metcalf, A. L. Fellows, E. C. van Diest, H. H. Logan, President Ridgway, and others.

Adjourned.

Atlanta Association

On March 14th, 1912, the Atlanta Association of Members of the American Society of Civil Engineers was organized, with the following officers: Arthur Pew, President; William A. Hansell, Jr., Secretary; and Messrs. James N. Hazlehurst and B. M. Hall, Members of the Executive Committee. The Association will hold its meetings in the house of the University Club.

Philadelphia Association

On December 22d, 1913, the Philadelphia Association of Members of the American Society of Civil Engineers was organized, with the following officers: George S. Webster, President; Richard L. Humphrey and F. Herbert Snow, Vice-Presidents; J. W. Ledoux, Edgar Marburg, and H. S. Smith, Directors; S. M. Swaab, Treasurer; and W. L. Stevenson, Secretary. The meetings of the Association will be held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

(Abstract of Minutes of Meeting)

January 27th, 1914.—The meeting was called to order; Vice-President Turner in the chair; Charles J. McGonigle, Secretary; and present, also, 20 members and 2 guests.

The minutes of the preceding meeting were read and approved.

A communication from Charles Warren Hunt, M. Am. Soc. C. E., Secretary of the American Society of Civil Engineers, was read.

The Secretary was instructed to ascertain the sentiment among the membership as to an amendment to the Constitution looking to changing the name of the Association to the "Portland, Oregon, Association of Members of the American Society of Civil Engineers," which change was suggested by Mr. Hunt, the Secretary of the Society.

On motion, the Secretary was instructed to obtain prices for printing the Constitution and By-Laws of the Association in pamphlet form.

A paper by Frederick C. Schubert, M. Am. Soc. C. E., on the "Dalles-Celilo Canal", was presented by the author, and the subject was generally discussed by those present.

Adjourned.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year: Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

Southern California Association

On January 5th, 1914, the Southern California Association of Members of the American Society of Civil Engineers held its first meeting at the University Club, Los Angeles, Cal., and elected the fol-

lowing officers: J. B. Lippincott, President; Charles T. Leeds, First Vice-President; George S. Binckley, Second Vice-President; W. K. Barnard, Secretary; and Charles H. Lee, Treasurer.

About 60 members of the Society were present, and many others have signified their interest in the formation of the Local Association.

A paper on the "Thomas System of Reinforcing and Three-Hinged Arch Bridges", was read by W. M. Thomas, Assoc. M. Am. Soc. C. E., who illustrated his remarks with lantern slides, and certain features of the work were discussed by those present.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London. W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, Room 6, City Bank and Trust Company Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From January 6th to February 2d, 1914)

DONATIONS*

TECHNIQUE OF MODERN TACTICS:

A Study of Troop Leading Methods in the Operations of Detachments of all Arms. By P. S. Bond, M. Am. Soc. C. E., and M. J. McDonough, Assoc. M. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., illus., 344 pp. Menasha, Wis., George Banta Publishing Company, 1913. (Donated by the Authors.)

The purpose of this volume, it is stated, is to supply in compact form the help needed by instructors, or students, of problems of military tactics, in the applicatory method of study, not only as a textbook, but also as a guide in the preparation or solution of such problems. The organizations used by the authors, are said to be those of the American Service, but the tactical principles discussed are of general application and pertain to systematic organized warfare against a civilized foe. In their discussion of various methods and formations, the authors, it is stated, have endeavored to describe clearly all the principles which may be applicable and, at the same time, to give concrete illustrations in figures, distances, etc., of simple cases. The book is intended as a ready reference for the officer in charge of garrison schools, militia instruction, field manœuvres, war games, etc., as an instructor, critic, and guide for the officer of the army or militia compelled to study alone, and as a guide to officers preparing for promotion examination, at the Service Schools, etc. There is a short bibliography which it is said may be found useful for study and reference in connection with the subject. The Contents are: Organization of the U. S. Army; The Preparation and Solution of Practical Problems; Field Orders; Patrolling; Advance Guards; Rear Guards, Flank Guards; Marches, Change of Direction of March, Camps and Bivouacs; Convoys; Artillery Tactics; Cavalry Tactics; Outposts, Combat, Attack and Defense; Organization of a Defensive Position; Combat; A Position in Readiness; Sanitary Tactics; The Rifle in War; Notes on Division Tactics and Supply.

THE HOLLOW-TILE HOUSE.

By Frederick Squires. Cloth, 10¼ x 7½ in., illus., 208 pp. New York, The William T. Comstock Co., 1913. $2.50.

In this book, as stated on the title-page, the reader is introduced to hollow-tile in the making, is told how it is wrought into houses and is shown, by numerous illustrations, how these houses look and from what foreign sources their appearance is an heritage. The Chapter headings are: Tile-Making; Old Work Stucco; About Construction; Counting the Cost; The History of the Use of Hollow Tile for Houses; Architects' Tile Houses; Building the Other Man's House; Floor Building; Tile in Stucco Surfaces; Tricks of the Trade; Tile and Concrete, Partners; Texture and Scale; The Flat-Roofed House; An Interesting Experiment; The House of Three Inventions.

FOWLER'S MECHANICAL ENGINEER'S POCKET BOOK, 1914.

Edited by William H. Fowler. Sixteenth Annual Edition. Leather, 6¼ x 4 in., illus., 66 + 576 pp. Manchester, England, Scientific Publishing Co., 1914. 2 shillings 9 pence.

This Pocket Book contains more than 600 pages of information useful to the mechanical engineer, including miscellaneous Tables and Formulas, matter relating to Steam Boilers and Fittings, Fuels and Combustion, Steam Engines, Steam Turbines, Steam Tables, Valves and Valve Gear, Internal Combustion Engines, Hydraulics, Pumps and Pumping Arrangements, Gearing and Lubrication, Hoisting and Lifting Machinery, Iron and Steel, Metals and Alloys, Beams and Pillars, Springs, Chemistry, and Ventilation and Heating. The Index covers 28 pages.

PROGRESS AND PROSPERITY:

The Old World and Its Remaking Into the New—The Story of the Mediums of Development—The Building of Empires in America, The World's Wonderland. By William de Hertburne Washington, Assoc.

*Unless otherwise specified, books in this list have been donated by the publishers.

M. Am. Soc. C. E. Three-quarters Morocco, $9\frac{1}{2}$ x $6\frac{1}{4}$ in., illus., 32 + 887 pp. New York, The National Educational Publishing Co., 1911. $4.50. (Donated by the Author.)

The author's aim in this book, it is stated, has been to tell the story of man's material civilization by portraying the evolution of transportation and its wonderful effect on the progress and prosperity of the world. He describes in detail the organization, operation, and influence of land and water transportation systems, both in the United States and abroad, the accompanying conditions of civilization, and their direct economic effects on present-day problems, in simple and non-technical language, illustrating his text with many and rare drawings. The Contents are: Foreword; The Essential Forces Making for Progress and Prosperity; The Magnitude of the Railroad; The Making of Nations; The Greatest Revolution of the World; Artificial Waterways; The Beginning of the Real New World; The Real Winners of the West; The Birth of a Railroad; Banks, Banking and Money; The Origin and Analysis of the Corporation; Capitalization and Overcapitalization; Cost of Construction of Steam Roads; The Analysis of the Cost of a Railroad; Analysis of the Profits; The Rich and the Poor Roads; Who are in the Business? Spending for Labor; Human Machinery; How the Railroad Business is Built Up and Maintained; Trusts and Trust Busting; Transportation a Developer and Creator of Land Values; Contrasts for Thoughtful Consideration; the Great Steam Liveryman and Public Truckman; Good Roads, Bad Roads, and Rail Roads; The Postal, the Express, and the Baggage Service; The Ocean Railway; Contributions to Progress and Prosperity by Other Modes of Movement; Railway Rates; Facts About Accidents; The Railroads of the World; Government Ownership; The Unseen Expenses of the Steam Roads; Caring for the Iron Horse, the Evolution of Equipment; Elimination of Natural Obstacles to Progress and Prosperity; Some of the Auxiliary Machinery; A Story of Real Progress and Prosperity; The Making of Empires; The New World's Future; Electricity *vs.* Steam; The Golden Age; Our Own the First Age of Real Progress and Prosperity; An Appreciation of the Future; Looking Forward; The World's Work; Index to Illustrations.

QUESTIONS AND ANSWERS

Relating to Modern Automobile Design, Construction, Driving and Repairs: A Self-Instructor for Students, Mechanics and Motorists. By Victor W. Pagé. Cloth, $7\frac{1}{2}$ x 5 in., illus., 15 + 622 pp. New York, The Norman W. Henley Publishing Company, 1914. $1.50.

In a secondary title, it is stated that this book is a practical treatise consisting of thirty-six lessons in the form of questions and answers, written with special reference to the requirements of the non-technical reader desiring easily understood explanatory matter relating to the construction, operation and repair of the automobile and including all the latest developments in automobile engineering. The text is fully illustrated with reproductions of actual engineering drawings and photographs of practical working parts. The subject is developed, it is said, in a logical manner, the reader being brought progressively from one element of the car to the next relative part, and the arrangement by lessons, it is hoped, will be found to be valuable as a textbook by those teaching automobile classes.

FOWLER'S MECHANICS' & MACHINISTS' POCKET BOOK & DIARY, 1914.

Edited by William H. Fowler. Sixth Annual Edition. Boards, 6 x 4 in., illus., 50 + 476 pp. Manchester, England, Scientific Publishing Co., 1914. 8 shillings.

The subject-matter contained in this volume, it is stated, includes a synopsis of practical rules for fitters, turners, millwrights, erectors, pattern makers, foundrymen, draftsmen, apprentices, students, etc., and contains exhaustive sections on Mensuration, Geometry and Trigonometry, Materials Used in Machine Construction, Machine Tool Design, Proportions of Machine Tool Parts, Metal-Cutting Tools, High-Speed Tool Steels, Drilling and Boring Metal, Screw Threads, Screw Cutting, and Taper Turning, Emery and Emery Wheels, Shop Practice, Wheel Gearing, Rope and Belt Driving, Shafting and Bearings, Lifting Ropes and Chains, together with numerous tables and an Index of 17 pages.

ROBERT FULTON, ENGINEER AND ARTIST:

His Life and Works. By H. W. Dickinson. Cloth, 9 x 6 in., illus., 14 + 333 pp. London, John Lane, The Bodley Head; New York, John Lane Company; Toronto, Bell & Cockburn, 1913.

In the preface the author states that his excuse for adding to the numerous biographies of Robert Fulton is that as far as he has observed none of the others

is altogether fair and impartial, his American biographers having credited him with greater achievements than facts warrant, and English writers have too often treated him as a charlatan, a filcher of other men's brains, and even as a traitor. The author states, therefore, that in this book he has endeavored by research and the collection of recently published material to preserve really important facts of Fulton's life and his inventions with sympathy and without bias. The Contents are: Ancestry, Birth, and Boyhood of Robert Fulton; Entry Into the World; Projects for Marine Propulsion; Correspondence with Lord Stanhope; Arrival in Paris; Second Attempt to Introduce the Submarine; Turns his Attention to Steam Navigation; Returns to England; Arrival in America; United States Patents; Construction of First Steam Man-of-War; Appendices; Index.

Gifts have also been received from the following:

Am. Concrete Inst. 1 pam.
Am. Inst. of Elec. Engrs. 2 bound vol.
Am. Ry. Master Mechanics' Assoc. 1 bound vol.
Am. Water Works Assoc. 1 pam.
Assam, India-Public Works Dept. 1 pam.
Bermondsey (London), England-Town Clerk. 1 bound vol.
Boston, Mass.-Dept. of Public Works. 1 bound vol.
Boston, Mass.-Transit Comm. 1 bound vol.
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British Columbia-Dept. of Lands. 1 pam.
Buckeye Engine Co. 1 pam.
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Cambridge Univ. & Town Water-Works Co. 1 pam.
Canada-Comm. of Conservation. 1 bound vol.
Canada-Dept. of Marine and Fisheries. 1 vol.
Canada-Dept. of Mines. 1 vol., 1 pam.
Canada-Dept. of Public Works. 1 vol.
Canada-Minister of Mines. 6 vol.
Canadian Inst. 2 pam.
Case School of Applied Science. 1 vol.
Chicago, Ill.-Bureau of Public Efficiency. 1 pam.
Columbia Univ.-Ernest Kempton Adams Fund. 1 vol.
Columbus, Ohio-Water Dept. 2 pam.
Connecticut-State Board of Health. 1 bound vol.
Darapsky, L. 1 pam.
Dartmouth Coll. 1 vol.
Dist. of Columbia-Engr. Commr. 1 vol.
Dooling, Peter J. 1 pam.
East Indian Ry. Co. 1 pam.
Germany-Preussische Landesanstalt für Gewässerkunde. 1 pam.
Glazier, William L. 2 pam.
Great Indian Peninsula Ry. Co. 1 pam.
Harvard Univ. 1 bound vol., 1 vol.
Hawaii-Supt. of Public Works. 4 vol.
Horton, Robert E. 1 pam.
Hyde, C. G. 1 pam.
Illinois Univ.-Eng. Exper. Station. 2 bound vol.
Institution of Civ. Engrs. 1 vol.
Institution of Naval Archts. 1 bound vol.
Johnson, Albert. 1 pam.
Jones, R. M. 1 pam.
Lowestoft, England-Town Clerk. 2 pam.
McCarty, R. J. 1 pam.
Massachusetts-State Board of Health. 1 bound vol.
Master Car Builders' Assoc. 2 bound vol.
Mellon Inst. of Industrial Research and School of Specific Industries. 1 pam.
Michigan-Secy. of State. 1 bound vol.
Minnesota Univ.-School of Mines Exper. Station. 1 bound vol.
Mississippi River Comm. 6 vol.
National Fire Protection Assoc. 4 pam.
New South Wales-Dept. of Mines. 5 bound vol.
New York City-Dept. of Parks. 2 bound vol.
New York City-Dept. of Water Supply, Gas, and Electricity. 1 pam.
New York City-Metropolitan Sewerage Comm. 2 pam.
New York State-Public Service Comm., First Dist. 1 bound vol.
New York-State Comptroller. 1 pam.
New York-State Engr. and Surv. 1 bound vol.
New York-State Library. 5 bound vol.
New York City Record. 2 bound vol.
New Zealand-Dept. of Mines. 12 vol.
Norwich Univ. 7 pam.
Pacific Tank & Pipe Co. 1 bound vol.
Panama R. R. Co. 1 pam.
Parker, Harold. 1 bound vol.
Permanent Inter. Assoc. of Navigation Congresses. 2 pam.
Philadelphia, Pa.-Bureau of Water. 1 vol.
Philippine Islands-Bureau of Health. 1 vol.
Philippine Islands-Bureau of Navigation. 1 pam.
Philippine Islands-Executive Bureau. 1 pam.
Polytechnic Inst. of Brooklyn. 1 pam.
Preston, England-Town Clerk. 1 bound vol.
Ramsgate, England-Town Clerk. 1 pam.
Reale Istituto Lombardo di Scienze e Lettere. 2 vol.
Rhode Island-State Board of Health. 1 bound vol.
Richmond, Fredericksburg & Potomac R. R. Co. 1 pam.
São Paulo, Brazil-Reparticão de Aguas e Esgotos. 2 vol.
Shanghai, China-Municipal Council. 4 vol.
Smith, J. Waldo. 1 bound vol.
Smithsonian Institution. 2 pam.
Southern Pacific Co. 1 pam.
Sunderland, England-Town Clerk. 2 pam.
Switzerland-Landeshydrographie. 1 bound vol., 1 pam.
Sydney, New South Wales-Harbour Trust Commrs. 1 pam.
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Tasmania-Geol. Survey. 14 pam.
Tasmania-Secy. for Mines. 1 vol.
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U. S.-Bureau of Lighthouses. 3 pam.
U. S.-Bureau of Mines. 6 pam.

U. S.-Bureau of Standards. 7 pam.
U. S.-Census Bureau. 7 bound vol., 2 pam.
U. S.-Chf. of Engrs. 3 bound vol.
U. S.-Coast and Geodetic Survey. 1 bound vol., 2 pam.
U. S.-Dept. of Commerce. 1 pam.
U. S.-Forest Service. 4 pam.
U. S.-Geol. Survey. 5 vol., 13 pam.
U. S.-Hygienic Laboratory. 3 pam.
U. S.-Patent Office. 1 vol.
U. S.-Secy. of the Interior. 2 pam.
U. S.-Weather Bureau. 2 bound vol.
Victoria-State Rivers and Water Supply Comm. 7 pam.
Virginia-Bureau of Labor and Industrial Statistics. 1 bound vol.
Virginia-Geol. Survey. 1 pam.
Worcester Polytechnic Inst. 1 vol.
Works, John D. 1 pam.

BY PURCHASE

Engineering and Metallurgical Books, 1907-1911: A Full Title Catalogue, Arranged Under Subject Headings, of all British and American Books on Engineering, Metallurgy, and Allied Topics, Published during the Five Years 1907-1911, with Their English and American Prices and Publishers' Names. By R. A. Peddie. D. Van Nostrand Co., New York, 1912.

Railway Problems. Edited, with an Introduction, by William Z. Ripley. Revised Edition. Ginn and Company, Boston, New York, Chicago, London, 1913.

The American Woods, Illustrated by Actual Specimens with Full Text. By Romeyn B. Hough. Part XIII, Representing Twenty-five Species by Twenty-five Sets of Sections. The Author, Lowville, N. Y., 1913.

Der Wirkungsgrad von Dampfturbinen-Beschauflungen. Von Paul Wagner. Julius Springer, Berlin, 1913.

Schwimmkörper aus Eisenbeton. Von Walther Stross. Wilhelm Ernst & Sohn, Berlin, 1911.

Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens. Herausgegeben vom Verein deutscher Ingenieure. Hefte 143-144. Julius Springer, Berlin, 1913.

Lehrbuch des Maschinenbaues. Von Karl Esselborn. Zweiter Band : Dynamomaschinen und Elektromotoren ; Hebemaschinen ; Baumaschinen ; Wasserkraftanlagen ; Bearbeitungsmaschinen ; bearbeitet von R. Bachmann, W. Kübler, W. Lindboe, A. Nachtweh, und H. Weihe. Wilhelm Engelmann, Leipzig und Berlin, 1913.

Jigs and Fixtures: A Reference Book Showing Many Types of Jigs and Fixtures in Actual Use, and Suggestions for Various Cases. By Fred H. Colvin and Lucian L. Haas. McGraw-Hill Book Company, Inc., New York and London, 1913.

Practical Uses of the Wave Meter in Wireless Telegraphy. By J. O. Mauborgne. McGraw-Hill Book Company, Inc., New York and London, 1913.

Principles of Industrial Organization. By Dexter S. Kimball. McGraw-Hill Book Company, Inc., New York and London, 1913.

Handbook for Machine Designers and Draftsmen. By Frederick A. Halsey. McGraw-Hill Book Company, Inc., New York and London, 1913.

Egyptian Irrigation. By *Sir* W. Willcocks and J. I. Craig, With an Introduction by *Sir* Hanbury Brown. Third Edition. 2 Vol. Spon & Chamberlain, New York ; E. & F. N. Spon, Limited, London, 1913.

Neuere Bogenbrücken aus Umschnürtem Gusseisen. Von Fritz Edler von Emperger. Wilhelm Ernst & Sohn, Berlin, 1913.

Barker on Heating: The Theory and Practice of Heating and Ventilation. By A. H. Barker. The Carton Press, London, 1912.

Poors' Manual of the Railroads of the United States, 1914. Poors' Railroad Manual Co., New York.

Handbuch der Ingenieurwissenschaften: Funfter Teil, Der Eisenbahnbau : Sechster Band, Betriebseinrichtungen Anhang die Kraftstellwerke ; bearbeitet von M. Gadow. Von F. Loewe und H. Zimmermann. Wilhelm Engelmann, Leipzig und Berlin, 1913.

Forscherarbeiten auf dem Gebiete des Eisenbetons: Der Einfluss der Längs und Querkräfte auf Statisch Unbestimmte Bogen-und Rahmentragwerke. Von B. Rueb. Heft 22. Wilhelm Ernst & Sohn, Berlin, 1914.

SUMMARY OF ACCESSIONS

(From January 6th to February 2d, 1914)

Donations (including 14 duplicates)	262
By purchase	19
Total	281

MEMBERSHIP

ADDITIONS

(From January 8th to February 5th, 1914)

MEMBERS		Date of Membership.
BOARDMAN, HAROLD SHERBURNE. Dean, Coll. of Technology, Univ. of Maine, Orono, Me	Assoc. M. M.	Feb. 3, 1904 Dec. 31, 1913
CODDINGTON, SAMUEL CECIL. Pres. and Engr., Coddington Eng. Co., 33d and Villard Ave., North Milwaukee, Wis		Dec. 31, 1913
ERICSON, ERIC GUSTAF. Prin. Asst. Engr., Northwest System, Penn. Lines W. of Pitts., Room 1121, Pennsylvania Station, Pittsburgh, Pa		Dec. 31, 1913
EVERETT, PERCIVAL HERBERT. Chf. Engr., Kern County Highway Comm., Room 109, Court House, Bakersfield, Cal		Dec. 31, 1913
GUDE, ALBERT VALDEMAR, JR. Engr. and Contr. (Gude & Co.), 712 Grant Bldg., Atlanta, Ga	Assoc. M. M.	June 5, 1907 Dec. 31, 1913
JAMES, EDGAR AUGUSTUS. (James, Loudon & Hertzberg), 26 Briar Hill Rd., Toronto, Ont., Canada		Dec. 3, 1913
MUKASA, SEITARO. Engr. in the President's Secretariat, Imperial Govt. Rys., 20 Fujimicho, Azabu, Tokyo, Japan		Dec. 3, 1913
PEASE, FLOYD ODELL. Care, Bolivia Development & Colonization Co., 9 rue Louis le Grand, Paris, France		Oct. 1, 1913
RANDORF, CHARLES ANDREW. Structural Engr., Lackawanna Steel Co., Buffalo, N. Y.	Assoc. M. M.	Dec. 6, 1910 Dec. 3, 1913
SMALL, CHARLES CHURCHILL. Box 888, Douglas, Ariz		Dec. 31, 1913
SMITH, WILLIAM ERNEST. Care, City Engr.'s Office, St. Paul, Minn	Assoc. M. M.	Dec. 1, 1908 Oct. 1, 1913
SPIKER, WILLIAM CLARE. Cons. Engr., 1504 Hurt Bldg., Atlanta, Ga	Assoc. M. M.	June 5, 1907 Dec. 31, 1913
STANTON, FRED CASWELL. Asst. Engr., Isthmian Canal Comm., Cristobal, Canal Zone, Panama	Assoc. M. M.	April 1, 1908 Dec. 31, 1913
WOODBURY, WILLIAM HOOK. With C. P. Ry., Kamloops, B. C., Canada		Dec. 31, 1913

ASSOCIATE MEMBERS

ADAMS, MILTON JEWELL. Chf. Engr., Hawaii Loan Fund Comm., P. O. Box 497, Hilo, Hawaii	Dec. 31, 1913
BAKER, ALBERT READ. Engr., Marin Municipal Water Dist., San Rafael, Cal	Oct. 1, 1913

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
BUMANN, CECIL SPENCER. Chf. Engr., Van Sant-Houghton Co., 503 Market St., Room 406, San Francisco, Cal..		Dec. 31, 1913
COOKE, CHESTER. Cons. Engr. (von Unwerth & Cooke), 609 East 9th St., Kansas City, Mo................		Nov. 12, 1913
COOPER, GILBERT KENYON. Acting Chf. Engr., United Fruit Co., of Costa Rica, Port Limon, Costa Rica........		Dec. 3, 1913
CRUM, ROY WINCHESTER. Asst. Prof., Experimental Eng., Iowa State Coll. of Agri. and Mechanic Arts, Ames, Iowa..		Dec. 31, 1913
FRANCIS, HOWARD LEWIS. 6ª Balderas No. 79, City of Mexico, Mexico................................		Oct. 1, 1913
GIVAN, ALBERT. City Engr., City Hall, Sacramento, Cal..		Nov. 12, 1913
HARRINGTON, ARTHUR WILLIAM. Dist. Engr.'s Office, U. S. Geological Survey, Boise, Idaho..............................	Jun. Assoc. M.	Dec. 6, 1910 Dec. 31, 1913
HART, RICHARD AMBROSE. Drainage Engr., U. S. Dept. of Agriculture, 319 Federal Bldg., Salt Lake City, Utah..		Dec. 3, 1913
JORDAN, MYRON KENDALL. Draftsman, Kansas City Structural Steel Co., 1317 South 33d St., Kansas City, Kans...........	Jun. Assoc. M.	Mar. 1, 1910 Dec. 3, 1913
LAUER, MARTIN PHILIPPE. Archt. and Engr. (Lauer & Young), 19 I. O. O. F. Bldg., Akron, Ohio..........		Oct. 1, 1913
LETTON, HARRY PIKE. San. Engr., U. S. Public Health Service, 1437 Clifton St., N. W., Washington, D. C................	Jun. Assoc. M.	Feb. 1, 1910 Dec. 3, 1913
LOOMIS, LESLIE BROWN. Asst. Chf. Engr., Transmission Tower Dept., Miliken Bros., Inc., Hillside Ave., Chatham, N. J..................................		Dec. 31, 1913
LYON, FREDERIC WILLIAM. Div. Engr., Bureau of Water, 529 Oliver Bldg., Pittsburgh, Pa.........................	Jun. Assoc. M.	Oct. 3, 1911 Dec. 31, 1913
McCLURE, HARRY CLIFFORD. Chf. Engr., Dept. of Architecture, Board of Education, 119 Prescott St., Toledo, Ohio.........	Jun. Assoc. M.	Mar. 1, 1910 Sept. 3, 1913
McCURDY, GEORGE EARLE. 3437 Berkley Ave., Berwyn, Ill..		Sept. 3, 1913
MARSH, CHARLES REED. Supt. of Constr., U. S. Public Bldgs., Treasury Dept., Long Branch, N. J.........................	Jun. Assoc. M.	June 30, 1910 Dec. 31, 1913
MARTIN, JOSEPH PATRICK. Dist. Engr., U. S. Forest Service, Ogden, Utah..................................		Dec. 31, 1913
MILLER, ALEXANDER NORMAN. Engr., The Hinman Hydr. Mfg. Co., 1060 Milwaukee St., Denver, Colo........		Dec. 31, 1913
MILLS, ADELBERT PHILO. Asst. Prof. of Civ. Eng., Coll. of Civ. Eng., Cornell Univ., Ithaca, N. Y.....................	Jun. Assoc. M.	Sept. 3, 1907 Sept. 3, 1913

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
PAYNE, JAMES ELWOOD. Sales Engr., General Fireproofing Co., 1435 Wick Ave., Youngstown, Ohio	Jun. Assoc. M.	June 6, 1911 Dec. 31, 1913
PEVERLEY, RALPH ST. LAWRENCE. Engr. and Supt., Snare & Triest Co., P. O. Box 1197, San Juan, Porto Rico		Dec. 3, 1913
RICE, ROWLAND GRENVILLE. 630 Brown Marx Bldg., Birmingham, Ala	Jun. Assoc. M.	Mar. 5, 1907 Dec. 3, 1913
STAFFORD, FREDERICK DIAL. Res. Engr., Hales Bar Lock and Dam, Guild, Tenn		Dec. 31, 1913
STILES, ALBERT IRVINE. 902 Manteo St., Norfolk, Va	Jun. Assoc. M.	Feb. 6, 1906 Dec. 31, 1913
STOCKER, EDWARD CHARLES. Chf. Hydrographer, Whangpoo Conservancy, Box 651, American P. O., Shanghai, China	Jun. Assoc. M.	April 4, 1911 Nov. 12, 1913
TEETER, EARLE EVERETT. Junior Engr., U. S. Reclamation Service, Las Cruces, N. Mex		Dec. 31, 1913
VEATCH, NATHAN THOMAS, JR. Prin. Asst., Worley & Black, 301 Reliance Bldg., Kansas City, Mo	Jun. Assoc. M.	Nov. 8, 1909 Oct. 1, 1913
WALKER, EDWARD GEORGE. 337 Beverley Rd., Hull, England	Jun. Assoc. M.	April 2, 1907 Dec. 3, 1913
WENZELL, ANDREW PERRY. Asst. Engr., C. P. Ry., Constr. Dept., Windsor Depot, Montreal, Que., Canada		May 7, 1913

JUNIORS

BROWN, GEORGE FRANKLIN. Care, Longview Farm, Lee's Summit, Mo	Nov. 12, 1913
CAMPBELL, CHARLES CECIL. Highway Insp., Bureau of Highways, Dept. of Public Works, 870 North Ringgold St., Philadelphia, Pa	Dec. 31, 1913
DALY, ALBERT PETER VINCENT. Care, University Club, Altoona, Pa	Dec. 31, 1913
DRURY, WALTER RHODES. 815 East 2d St., Flint, Mich	Dec. 31, 1913
FEINER, MARK ANTONY. 3143 Broadway, New York City	Dec. 3, 1913
HATCHER, THOMAS VICTOR. Care, U. S. Reclamation Service, Fort Shaw, Mont	Dec. 3, 1913
MELIN, OSCAR WILLIAM. 1014 Sixteenth Ave., Moline, Ill	Dec. 3, 1913
MORSE, FREDERICK THURLOUGH. 225 South Glendale Ave., Tropico, Cal	Nov. 12, 1913
SQUIBB, GEORGE SAMPSON. 12 Kilsyth Rd., Brookline, Mass.	July 2, 1913
STAEHLE, GILBERT COBB. Engr. in Chg., Drafting Room, Concrete Eng. Co., 608 Omaha National Bank Bldg., Omaha, Nebr	Dec. 3, 1913
WEBB, CHAUNCEY EARL. Mason, Mich	Dec. 31, 1913

RESIGNATIONS

MEMBERS	Date of Resignation.
FORD, PORTER DWIGHT	Dec. 31, 1913

DEATHS

ARANGO, RICARDO MANUEL. Elected Associate Member, September 2d, 1896; Member, February 6th, 1906; died January 24th, 1914.

HAYS, JOHN WILLIS. Elected Member, June 5th, 1901; died December 14th, 1913.

MIX, EDGAR HENRY. Elected Associate Member, April 2d, 1913; died January 7th, 1914.

PETERSON, PETER ALEXANDER. Elected Member, January 5th, 1876; date of death unknown.

RIEGNER, WALLACE BERKLEY. Elected Member, September 7th, 1904; died January 19th, 1914.

Total Membership of the Society, February 5th, 1914, 7 285.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(January 6th to February 1st, 1914)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Institute Indicator*, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *Engineering Record*, New York City, 10c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *Railway Age Gazette, Mechanical Edition*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer*, Scranton, Pa., 25c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.

(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(71) *Journal*, Iron and Steel Inst., London, England.
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*Illustrated.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XL. FEBRUARY, 1914. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

By John C. Koch, Assoc. M. Am. Soc. C. E.

To be Presented March 4th, 1914.

Introduction.

The writer has been investigating the subject of sand-clay road construction for the past 2 years, for the purpose of establishing a sound basis for the selection of suitable materials for use in this cheap type of road. The materials are so cheap and distributed so abundantly that the writer is convinced that, if the Engineering Profession realized more fully the many advantages of such roads, there would be a much wider adoption of this type of construction.

The purpose of this paper is to set forth the results of the writer's study of these materials, both in the field and in the laboratory. This work was done under the auspices of the University of Georgia, a Good Roads Department having been established in the fall of 1911 for the purpose of carrying out a policy of University Extension work by offering free engineering assistance to the road officials of the State on highway and bridge work. The writer makes grateful acknowledgment of the facilities placed at his disposal by Chancellor

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

D. C. Barrow, of the University of Georgia, and C. M. Strahan, M. Am. Soc. C. E., for many helpful suggestions.

The growing demand for improved country roads at a low enough first cost to permit the construction of a relatively large mileage in counties of average financial resources has led, in many scattered sections of the United States, to an extensive adoption of the sand-clay type of construction.

Sand and clay are the most widely distributed materials in Nature, and, in suitable mixtures, yield a road-surfacing material which can be secured at low cost, because so readily accessible, and when properly built of intelligently selected material, it will render excellent service. Briefly stated, the advantages of sand-clay mixtures for road surfacing are:

(1) Low first cost,
(2) Low maintenance,
(3) No expensive machinery for construction or repairs,
(4) No skilled labor required.

Mixtures of sand and clay in proper proportions for road surfacing often occur in Nature, though good results can be secured by making artificial mixtures. The name "topsoil" has been widely used in Georgia and other parts of the South to indicate a natural mixture of sand and clay found on the surface of the ground over wide areas in the Appalachian region. Such natural mixtures, doubtless, are to be found in many other parts of the United States.

The principal advantages in using natural mixtures of the proper proportions, as compared with artificial ones are:

(1) The expense of mixing is eliminated,
(2) The natural mixtures become compacted in much less time,
(3) Repairs are more easily made, and new materials unite more quickly to the old.

The firmness of the wet sand road and the smoothness of the dry clay road are combined in the sand-clay type, and it remains of almost uniform smoothness throughout the year. It has been known for years in many localities that there were stretches of earth road that always kept hard and smooth, shed the water, and scarcely ever needed any repairs. Fig. 2 illustrates a section of natural sand-clay road near Center, Ga., which has been in use for 60 years. In such places

the surface soil has been found to contain the proper proportions of sand and clay, which, remaining undisturbed, has become compacted into a hard, water-proof surface.

There are many miles of sand-clay roads in Georgia which have cost less than $500 per mile for the surfacing in place. They carry heavy country traffic, and the repairs, during a period of 5 years, have not averaged $5 per mile per year. Considering the fact that the first cost of construction of such roads is less than the annual interest and maintenance charges on almost every other type which has been offered as an improvement on the earth road, it seems inevitable that wherever sand-clay materials are available and of suitable quality, it is the best type to use for the average country road.

Objects of the Investigation.

The writer's objects were to determine:

(1) The proper proportions of sand and clay in sand-clay mixtures to yield the best results, namely, durability in all weathers and absence of mud and dust;

(2) The limits between which the proportions of the two materials could vary without affecting seriously the quality of the resulting road surface;

(3) The effect of varying proportions of different sizes of sand in such mixtures;

(4) An approximate field method for the examination of soils, to determine their suitability for road surfacing.

Basis of the Investigation.

It will be readily seen that, to carry out an investigation along these lines, using artificially proportioned mixtures on short lengths of experimental road, would not only involve large expense, but probably would not be conclusive because of the difficulty of making artificial mixtures according to given proportions and maintaining such proportions. Then again, materials tracked on the experimental sections by the traffic would tend to affect greatly the value of such sections. For these reasons it was determined to base these studies on roads already constructed in various parts of Georgia, and in service from 1 to 5 years. In the course of this work the writer has traveled over 2 500 miles of roads, selecting various sections of sand-clay (including

"topsoil") roads which had been in use for some time and had worn well, and concerning which definite information could be secured as to materials used, methods of construction, and behavior of the road in all seasons of the year. From the best sections selected, samples of the road surfacing in actual service were chopped out with a hatchet for a depth of 4 in. in the wheel-tracks. Usually, a 5 by 5 by 4-in. sample was taken for laboratory study.

Samples were also taken from sections where such roads had not given entire satisfaction, on account of dust and mud. The writer has examined more than 900 samples, representing 48 counties, and comprising every typical sand-clay section of the State.

Properties of the Materials.

Clay.—The properties of a clay depend on the kaolin minerals and the impurities it contains. The kaolin particles may consist of prismatic crystals, thin plates or scales, or both. The effect of the impurities depends on the quantities of each and its fineness. Clays have no characteristic color; the pure are white, but they commonly occur as blue, gray, red, yellow, green, and purple, due to impurities.

All clays are derived from the weathering of feldspar, and the particles vary in size from small pebbles to such a minuteness as to defy measurement by the most refined methods. The finer particles readily go into solution in water, and even the larger ones are easily moved by running water. When removed in this manner and deposited at a distance, the deposits formed are called sedimentary clays. Those formed in place by the weathering of feldspar are called residual clays. In general, the sedimentary are finer grained and more plastic than the residual clays. The writer's experience has been that the latter make a harder and tougher mixture for road work than the sedimentary clays when combined with sand.

Clays vary greatly in density, plasticity, and size of grain. The important characteristics of this material for sand-clay construction are plasticity, shrinkage, and the property of "slaking". The plasticity is indicated by the ease with which a clay when wet to a certain extent can be moulded into various shapes which will be retained after the material has dried. The drying out of clay produces a shrinkage which seems to consolidate the particles into a compact mass. The extent of the shrinkage will depend on the fineness of the clay particles.

The coarser clays and those with much impurity (sand, etc.) shrink but little. The purer clays show a tensile strength of from 50 to 200 lb. per sq. in. when thoroughly dried; the coarser and impure ones have very little tensile strength. The most plastic clays resist water for a long time, but the others often crumble to pieces like quicklime. This rapid breaking down is due to the absorption of water by the pores of the clay, sometimes so quickly as to act with almost explosive suddenness. It is evident that a slaking clay would tend to break down rapidly into mud when exposed to rain and the puddling effect of traffic.

Sand.—Sand consists of particles of various sizes formed by the disintegration of the many kinds of rock in which quartz appears as an important component. As ordinarily found, the individual particles possess great crushing strength, as well as resistance to abrasion. These two qualities are of the greatest importance in the sand-clay combination.

Silt and Organic Matter.—In most of the natural sand-clay mixtures occurring in proper proportions for use directly as a surfacing material, the quantity of silt is usually from 3 to 8 per cent. The quantity of organic matter varies considerably, but, as it is light and easily washed out of the road by rains, its only effect seems to be to hold a certain quantity of moisture beneath the surface. Silt, if not in too large a quantity, seems to improve the road. In a few cases the writer has found silt and organic matter together in quantities as great as 20% of the total sand-clay mixture, and yet the road was good. Probably in such cases a large quantity of water is held by this porous material and this aids in binding the sand particles together. Thus it appears that these materials are not necessarily harmful to the mixture.

MIXTURES OF SAND AND CLAY.

Whether natural or artificial mixtures are used, the properties of both materials which are best adapted to resist the weather and the wearing effect of traffic are utilized, and the unfavorable qualities of both are largely eliminated by using the two in proper combination. It has been shown that clay in the pure state shrinks as much as 10% when dried, and on being wet again will expand to an equal extent. By mixing clay with the proper quantity of sand, these contractions and expansions may be eliminated almost entirely. There-

fore, in a sand-clay mixture for road work, the sand, being held together by the clay, which usually more than fills the voids in it, takes the wear of the traffic in dry weather. In wet weather the water quickly drains off this almost impervious mixture, and the sand resists the cutting action of the traffic, although the clay may become puddled to a certain extent.

The writer is aware of the difficulties and intricacies introduced into the subject by the wide variety of clays available and their different characteristics. Nevertheless, he believes that the methods given in this paper, which have thus far proved highly satisfactory in the selection of sand-clay materials in many parts of Georgia, will be of general value wherever such materials are to be found. This basis of study of these materials has been of great service in choosing the best of a number of equally accessible deposits of natural sand-clay mixtures. It may with equal facility be used for determining in just what proportions the different materials should be used in order to get the best mixture possible out of the available local deposits where it is necessary to make artificial mixtures. Most clays contain varying proportions of sand, even when they appear to be almost pure. Clays are often sent in from the southern part of Georgia (where the surface soil is very sandy and it is desirable to lay clay on top of it in order to mix with it and form a sand-clay surface), and on analysis are found to contain as much as 60% of sand.

Laboratory Procedure.

Equipment.—So much that is useless is often purchased for laboratory equipment that the following list is given for the benefit of those who seek definite information on this subject. These have been found to give quite satisfactory service, and need not cost more than about $50.

One metric balance, 111-gramme capacity, reading to 0.01 gramme.

One metric scale, 2 500-gramme capacity, reading to 5 grammes.

One set of sand sieves, nesting, 8 in. in diameter, Nos. 10, 20, 40, 60, 80, and 100, with dust pan and lid, all brass.

Two steel moulds, brass lined, 1 in. in diameter, 4 in. long, with 5-in. close-fitting plunger.

One wooden mallet.

Six 300-cu. cm. evaporating dishes, porcelain.

Two Bunsen burners.
Several small tripods for burners.
Several iron wire mats, 4 by 4 in., for Bunsen flame.
Thermometer, 0° to 150° cent.
Six, 2-quart, enameled milk pans for baths, etc.

The evaporating dishes should be Royal Berlin porcelain, as this is the only kind that the writer has found satisfactory.

Separation of Materials.—Samples are usually taken in several parts, so as to enable one to determine to what depth the material may be used. In some cases a sample may be in three or more parts, being 4-in. layers taken in succession from the same hole, each being kept separate from the others. After analyzing each part of a sample separately, the composition of the top 4, 8, 12, or 16 in. can be determined easily. This gives a clear idea of the value of the material and of the greatest depth to which it may be excavated.

The essential objects of the laboratory examination are to ascertain:

(1) Relative proportions of sand and clay,
(2) Physical analysis of sand,
(3) Physical properties of the mixture of sand and clay,
(4) Determination of presence of other material which might prove detrimental,
(5) Physical properties of the clay.

To attain these objects, the following steps are necessary, in the order of their importance, and to reduce the laboratory work to a minimum:

(1) Separation of sand and clay,
(2) Mechanical analysis of the sand content,
(3) Slaking test on cylinder of sand-clay,
(4) Examination for mica and feldspar,
(5) Slaking test on clay cylinder.

1.—Separation of Sand and Clay.—The sample to be examined is first dried thoroughly in the air; then it is screened through a No. 10 sand sieve (10 meshes per linear inch), lumps and clods being pulverized with a wooden mallet beforehand. All coarse materials caught on the No. 10 sieve are considered arbitrarily as gravel, and all smaller materials are taken as the sand-clay portion. Pieces of grass, roots, etc., are discarded, of course, before weighing the two parts of the

screened sample. The material held on the No. 10 sieve is regarded as being much the same as the broken stone in a concrete mixture, and the sand-clay as the cement and sand portion constituting the mortar in such a mixture.

The part of the sample passing the No. 10 sieve is then examined as follows: By successive quartering, about 150 grammes are taken and dried in the air bath at 100° cent. to constant weight. This may often be dispensed with, and the sample merely air dried. Then 100 grammes of this material are weighed and placed in a porcelain evaporating dish; water is added and the soil rubbed well until the clay particles are in suspension. The clay in suspension is then carefully poured out and more water is added; the process is repeated until there is faint or no coloration of the water when the residue is stirred up. By washing the materials properly, practically all the organic matter, silt, and clay are removed, and only the sand particles are left, with possibly some mica and feldspar.

2.—Mechanical Analysis of the Sand.—The moisture in the residue in the evaporating dish is then evaporated, leaving the sand dry and ready to be screened. After weighing the sand it is screened through a nest of five sieves having 20, 40, 60, 80, and 100 meshes per linear inch. The weights of sand caught on each of these sieves are determined, and a curve is plotted showing the weights as ordinates and the sieve sizes as abscissas, based on 100% for the total weight of the mixture of sand and clay. Thus the weights in grammes show as percentages of the total. Fig. 1 was prepared in this way, and shows graphically the sand analyses of a number of samples.

3.—Soil Slaking Test.—A test cylinder, 1 in. in diameter and 3 in. long, is made of the material passing the No. 10 sieve by wetting it sufficiently to make a very stiff paste and after working it thoroughly together, placing it in a metal mould, using a tight-fitting plunger to compact the material as much as possible by tamping with a mallet. This cylinder should be dried to constant weight in the air bath at 100° cent. When entirely cooled it should be immersed completely in a glass jar of water at a temperature of 21° cent., and the time noted in which it disintegrates completely. Disintegration is assumed to be complete when the cylinder has broken down until the material is standing approximately at its natural slope of repose.

In sand-clay mixtures which have given satisfactory service, the

time to disintegrate completely may vary from 2 min. to nearly 1 hour; usually, it will be from 5 to 20 min. This test will give a fairly good idea of the resistance of any sand-clay mixture to the action of water, and, for purposes of comparison, is made more easily and

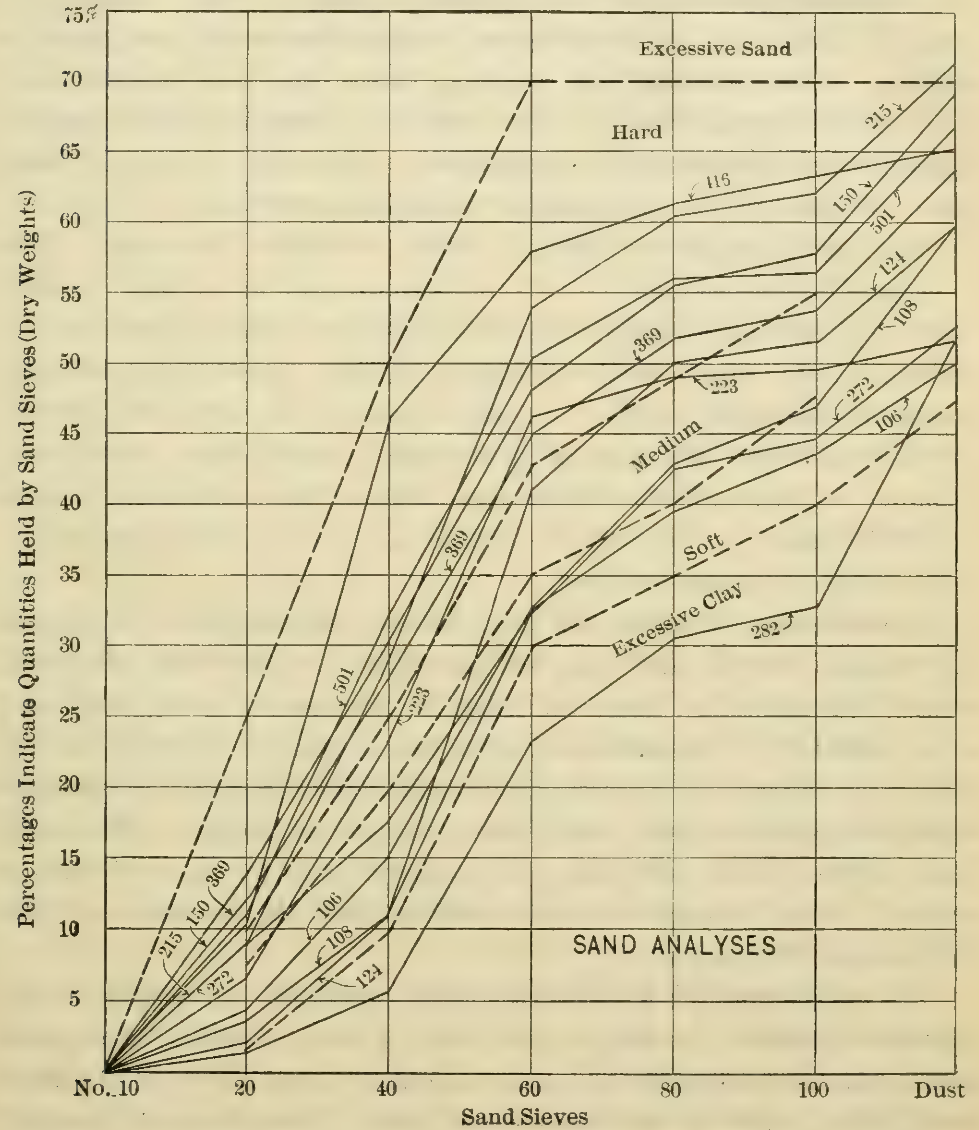

FIG. 1.

quickly than that to be described for the slaking of clay alone. The most durable mixtures, in general, are those which take longest time to disintegrate. Mixtures containing a coarse, flaky clay may hold their shape for several days before breaking down completely, but this is not a frequent occurrence.

4.—Tests for Mica and Feldspar.—Mica is easily recognized, and its presence need occasion no alarm unless it exceeds 5% of the total sand-clay sample. This is especially true of the Southern Appalachian region, where mica is often associated with the coarser sands, as in the weathered gneisses of the crystalline areas of that region. In excess of about 5%, mica will cause considerable trouble, acting as a lubricant in wet weather and crushing into a fine powder when dry. It is easily detected with the low power of a compound microscope, a magnification of from 30 to 50 diameters being large enough, in most cases, to make out the characteristic laminated structure.

The separation of the mica is often a difficult operation. This is especially true of soils in which it is present in a finely divided state. In washing a sample of such a soil, the mica will go into suspension in the water with the clay and silt. If the clay and silt precipitate before the mica, it is easy to remove the latter by drawing off the wash-water and passing it through a paper filter. Otherwise, it would be impossible to effect a separation of these materials. Soils in which there is much finely-divided mica will usually present a characteristic, lustrous surface.

In most cases, however, the mica can be removed from the sample by washing out the clay and silt carefully (as previously described under "Separation of Sand and Clay"), the residue being sand and mica particles. A separation of these can be readily effected, the difference in the specific gravities being sufficient to permit of a gravity separation when covered with water and gently shaken. The sand sinks to the bottom and the overlying mica can be removed with a spatula. After drying, the mica is weighed and the percentage of the total sample determined.

The most harmful of the feldspathic materials, according to the writer's experience, occurs in the form of small pebbles, or even in large irregular masses, of an earthy appearance, and of a color ranging from light yellow to dark brown or black. Though easily crushed, it will generally have more coherence than the clods of earth or clay in the sample. Therefore, the larger pebbles can be separated by screening the dried sample through the No. 10 sieve. If pebbles of other material are also found, the difference in color will permit of a separation. The feldspathic materials passing the No. 10 sieve, together with the sand, clay and silt, will be left with the sand, as a

residue after washing out the other materials. Separation of the feldspar can then be made in the manner described for mica. If this material occurs in greater proportions than about 8% of the total sample, by dry weight, the road surface will cut and wash easily. This is a figure based on actual experience with a few roads, and is probably subject to modification, depending on the characteristics of the other components of any given sample. The presence of fine feldspathic material will be readily apparent in the soil cylinder test. When the soil cylinder is immersed the feldspar will almost immediately go into solution or suspension, and in a minute or so the water will become so discolored that the cylinder cannot be distinguished.

The number of cases in which a separation of either of these materials is advisable will vary considerably. In the writer's experience, about 6% of all samples examined were analyzed quantitatively for mica or feldspar. Although the methods described are approximate, they have given very satisfactory results.

5.—Slaking Test on Clay Cylinder.—The clay from the 100-gramme sample can be collected by saving the wash-water and allowing the clay to settle. Usually, the quantity of clay from such a small sample is not sufficient to make the test cylinder, so that additional clay must be removed from the material passing the No. 10 sieve to make up the cylinder. This cylinder is made by mixing the dried clay with only sufficient water to make a very stiff paste, which is then moulded under sufficient pressure to make a compact mass. After drying in air and then to constant weight in the air bath at 100° cent., the specimen is kept at that temperature until at least an hour after the last trace of moisture is found by condensation on a glass plate above the cylinder. It is important that the cylinder be thoroughly dried. When properly dried it is completely immersed in a glass vessel of proper size containing water at 21° cent. The character of the slaking and the time required for complete disintegration depend somewhat on the temperature of the water, and the freedom from moisture of the dried cylinder, so that, for a comparison of results, all specimens should be thoroughly dried and the water for the test brought to the same temperature in each case.

The slaking of clay is similar to that of quicklime, except that the action is entirely physical and not chemical. The time and character

of the slaking give an idea of the value of the clay as a binder in the sand-clay mixture. In clays which have been separated from samples taken from road surfaces which have proved satisfactory, the time for the clay cylinder to disintegrate completely varies from 2 to 20 min., the average being about 4 min. The degree of disintegration of the clay may vary from finely divided particles to flakes almost ⅛ in. long. In general, the coarser the particles the better the indication for suitability for road work. Clays which disintegrate completely in less than 2 min., may be regarded with some suspicion, but need not necessarily be rejected entirely, unless the sand analysis and the soil cylinder also give poor tests.

Typical Examples, with Analyses and Road Histories.—It is thought that a few characteristic cases from actual practice may be of interest. In Table 1 is given a list of the samples, with the locality from which each was taken. Table 2 shows the sand and clay analysis, based on the dry weight of the sample, the total weight of each sample being taken as 100%; all sand percentages are based on this.

TABLE 1.—TYPICAL SAND-CLAY ANALYSES.

Percentages of Total Weight of Sand and Clay.

Sample No.	106	108	124	150	215	223	272	369	416	501	282
Sand No. 10	(12.5)	....	....	(6.3)	....	(34.0)	....	(7.2)	(1.5)	(10.0)	....
" No. 20	4.5	3.6	2.0	13.7	9.2	6.8	9.2	12.0	10.6	11.0	1.4
" No. 40	10.8	7.6	9.0	16.6	19.5	15.9	8.6	15.7	35.4	21.4	4.3
" No. 60	17.5	21.5	30.0	17.7	25.2	23.5	14.5	17.3	11.9	17.8	17.5
" No. 80	6.7	10.2	9.5	7.5	6.5	2.8	10.3	6.8	3.3	5.8	7.3
" No. 100	4.0	4.0	1.0	2.3	1.6	0.4	2.0	2.0	2.1	0.4	2.1
" Dust	6.5	13.0	8.5	11.5	9.5	2.1	8.0	10.0	2.0	10.4	19.1
Total Sand	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Clay	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3
Sand 20-60	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2

NOTE.—Sand No. 10 is based on gross sample = 100%, and includes all gravel. Remaining portion of table is an analysis of material smaller than No. 10.

Fig. 1 shows a graphic analysis of these samples, with the sand analysis in detail. This diagram also gives, graphically, the limits between which the writer classifies his samples, basing such classification on the sand analysis as well as the clay content, both of which are shown.

The limiting curves separating the analyses into four groups are based on the writer's experience, and are given with full confidence in their utility. They are of sufficient accuracy to be of considerable

service in classifying sand-clay mixtures properly and quickly, and arriving at a reasonable approximation of their value as road material. Quite often it will be found that the sand analysis alone may show that the material is unsuitable and much time may be saved that might otherwise have been lost in continuing the examination and applying other tests.

TABLE 2.—ROAD HISTORIES.

Sample Numbers Refer to Corresponding Numbers in Table 1.

Sample No.	County.	Road.	Sample taken.
106	Bulloch	Statesboro-Savannah	6.7 miles south of Statesboro.
108	Dougherty	Albany-Thomasville	4.3 miles south of Albany.
124	Sumter	Americus-Albany	6.0 miles south of Americus.
150	Clarke	Athens-Danielsville	3.2 miles northeast of Athens.
215	Dougherty	Albany-Thomasville	2.1 miles south of Albany.
223	Richmond	"Augusta gravel" from pit.	....
272	Habersham	Clarkesville-Cornelia	1.2 miles from Clarkesville.
369	Clarke	Athens-Barnet Shoals	4.3 miles from Athens.
416	Hall	Thompson's Bridge	2.0 miles north of Gainesville.
501	Clarke	Athens-Whitehall	2.0 miles from Athens.
282	Decatur	Bainbridge-Jacksonville	7.0 miles south of Bainbridge.

The analyses falling in the group marked "Hard" will usually give a durable, hard road surface which wears exceedingly well, and, after consolidation, can be cut only with great difficulty by a road machine, if at all. Analyses in the "Medium" group will give an excellent, smooth, hard, surface, but one which may cut a little in protracted wet weather. Such a surface is too hard to attempt to shape up with a road machine, except after very long wet periods. This may be taken as a sort of "average" quality of sand-clay.

The analyses grouped in the "Soft" class give a surface which is much superior to the ordinary earth road, but which will tend to be somewhat dusty in dry weather and will cut easily and tend to wash in wet weather. Material of this type requires re-shaping, after every heavy rain, with a road machine or drag. The class "Very soft" comprises all materials which have too large a clay content to give a satisfactory road surface; they should not be used at all, as the expense is scarcely warranted for the kind of road surface that can be obtained with it.

The writer's method is to make the separation of sand and clay and then the mechanical analysis of the sand. If the material shows up poorly in the sand analysis, the further tests are not made, unless,

after making a similar examination of all other available materials, it appears to be the best available. Each case is a special problem in economy, and a study of each will indicate the proper method of using local materials if they can be used at all.

Road Histories.

The following is a brief history of the various roads, samples of which were analyzed and the results presented in Table 1, and graphically in Fig. 1:

106.—Bulloch County.—Had been in service 3 years at time sample was taken. Excellent, hard, smooth surface, but occasionally dusty in prolonged dry seasons. Softens in wet weather and cuts badly then. Re-shaped after heavy rains with light drags. Artificial mixture.

108.—Dougherty County.—Good smooth surface in dry weather. Road in service about 3 years at time sample was taken. Softens and washes in heavy rains. This material, found about 2 ft. below natural surface, is used without addition of other material.

124.—Sumter County.—Very good, smooth surface, but softens and washes badly in wet weather. Re-shaped with light drags after even moderate rains. This is generally true of all the roads in this county, the materials available being very similar in mechanical analyses. Natural mixture occurring 2 ft. below surface.

150.—Clarke County.—Road in use 4 years, with practically none but the very lightest repairs, when sample was taken. Cost of maintenance, as given by superintendent, averaged less than $5 per mile per year. Very little dust, firm, hard road surface throughout the year, though freezing and thawing might at times soften the crust for a depth of an inch or so. No re-shaping with road machine possible, on account of the hardness of the compacted material. This seems to represent the best and most durable type of sand-clay combination yet found in Georgia. Natural mixture found as topsoil of cultivated fields.

215.—Dougherty County.—Road in use about 3 years at time sample was taken. Good hard surface, softens but little in wet weather, and wears well, keeping good shape. Too hard to re-shape by dragging. Few repairs. Natural topsoil.

223.—Richmond County.—Reports of roads on which this material had been used indicate that it produces very satisfactory results. This

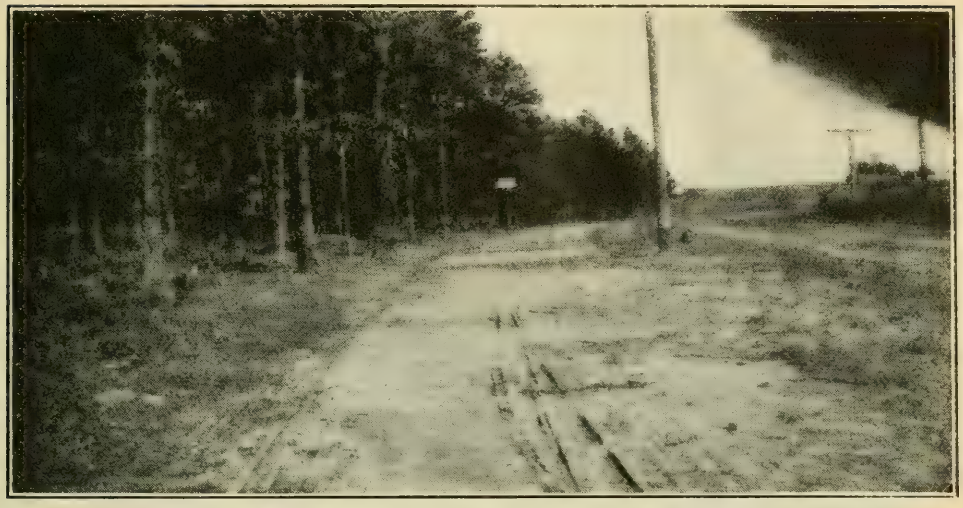

FIG. 2.—NATURAL SAND-CLAY ("TOPSOIL") ROAD, NEAR CENTER, GA. IN USE ABOUT 60 YEARS.

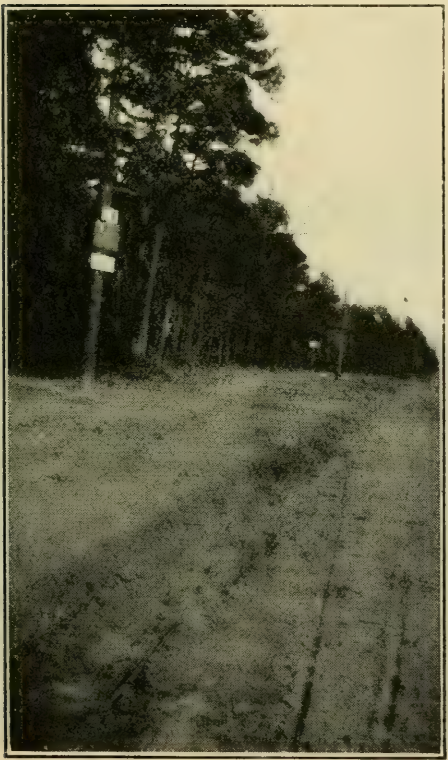

FIG. 3.—NEAR THE SAME PLACE AS SHOWN BY FIG. 2. NEW ROAD ON THE RIGHT.

FIG. 4.—TYPICAL SAND-CLAY ROAD IN THOMAS COUNTY, SOUTHERN GEORGIA.

FIG. 5.—TALLASSEE ROAD, CLARKE COUNTY, GEORGIA. TOPSOIL IN PLACE SIX DAYS, WITH TWO DAYS OF HEAVY RAIN.

FIG. 6.—CENTER ROAD, CLARKE COUNTY, GEORGIA. CLAY ROAD AT LEFT. NATURAL TOPSOIL ROAD AT RIGHT.

material is shipped by rail for road construction for distances as great as 100 miles. The analysis in Table 1 indicates that this is really a low-grade gravel rather than a sand-clay mixture. Yet it is interesting to note that the mechanical analysis of the portion of this material smaller than No. 10 mesh agrees very well with the mechanical analysis of the better classes of sand-clay mixtures which, with very small gravel contents, have given results almost as good. The mechanical analysis is shown on Fig. 1, considering only the portion smaller than No. 10 mesh. This material is dug from a large pit near Augusta, and has been in use for many years.

272.—Habersham County.—The road from which the sample was taken had been in use for about 8 months. The surface is hard and smooth, and softens but little in wet weather. This material was selected by the writer after sampling a large number of local deposits. Haul, about 2 000 ft. Cost in place, for 16-ft. width of surfaced roadway, about $500 per mile. Local officials, after using macadam at about $6 000 per mile, pronounce this a better road than macadam, even if initial cost and maintenance were equal. Local natural mixture of sand-clay from topsoil of cultivated field.

369.—Clarke County.—Built about 3 years at time sample was taken. Hard, durable surface. Few repairs. Too hard to drag. Experience quite similar to that of No. 150. Natural topsoil sand-clay mixture, obtained from nearby cultivated fields.

416.—Hall County.—In use less than a year. Replaced macadam road that cost $6 000 per mile, and gives better satisfaction than the macadam. Cost $350 per mile for 16-ft. width of roadway. Material natural topsoil from nearby cultivated field.

501.—Clarke County.—Road from which sample was taken had been in use about 3 years. This road was surfaced with a very thin layer of natural topsoil from adjoining fields, so that, after being compacted by traffic, its thickness was about 3 in. In seasons of prolonged wet weather traffic occasionally cuts through and mud-holes are formed. The material is excellent, but has not been used in sufficient quantity to give the best results. A comparison of its mechanical analysis with that of other samples indicates its high quality.

282.—Decatur County.—The road from which this sample was taken had been in use about one year. This is practically no improvement over the ordinary earth road, as the sand content is so low as

to be of no value in improving the quality of the road surface. It is very dusty in dry and very muddy in wet weather, after the traffic has been on it a few days. As difficult to maintain as an ordinary earth or clay road. Material used was taken from adjoining fields at a depth of 2 ft.

Table 3 is a summary of the road histories and the corresponding analyses, for comparison. For simplicity, the portion of the sample larger than No. 10 will be considered apart from that smaller than No. 10.

Effect of No. 10 Sand.—Comparing Sample No. 106 with the others, it is found to contain a larger quantity of No. 10 sand than any of them, with the exception of No. 223. No. 223 is really a low-grade gravel, and is discussed in a succeeding paragraph. The history of No. 106 is not nearly so satisfactory as that of the remaining samples, except Nos. 108 and 282. Sample No. 108 is slightly more satisfactory than No. 106, and No. 282 is practically an earth road. Furthermore, Samples Nos. 124, 215, and 272, which contain no No. 10 sand at all, have given much more satisfactory service than No. 106.

It has been found by the writer, in samples of gravel and chert roads (Augusta gravel being a typical material), that the part larger than No. 10 varied considerably in the few samples examined, although the composition of the part smaller than No. 10 was fairly constant in the proportion of sand to clay. The gravel part of any sample may reasonably be taken as an approximate measure of the durability of a road built with such mixtures. In addition, if a sample has a considerable proportion of gravel and is deficient in sand, the presence of the gravel will largely offset such defect.

From the foregoing it is clear that the effect of No. 10 sand on a mixture may be, either that of making up a deficiency of sand in the portion of the sample smaller than No. 10, or to increase durability when there is no deficiency of sand. Hence, its effect depends largely on the analysis of the portion of the sample smaller than No. 10.

Material Smaller than No. 10.—Based on their histories, the samples of Table 3 may be divided into four groups:

1. Ordinary earth, No. 282;
2. Inferior sand-clay, Nos. 106, 108, 124, and 272;
3. Superior sand-clay, Nos. 150, 215, 369, 416, and 501;
4. Low-grade gravel, No. 223.

TABLE 3.—SUMMARY OF ROAD HISTORIES AND ANALYSES.

Sample No.	106	108	124	150	215	223*	272*	369	416*	501	282*
Character of Road Service:											
In dry weather	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Medium
In wet "	Soft	Soft	Soft	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Soft
In prolonged weather	Cuts 2″–6″	Cuts 2″–4″	Cuts 1″–4″	Cuts ½″–1″	Hard	Hard	Cuts 1″–2″	Cuts ½″	Cuts ½″	Cuts ½″	Cuts 6″–10″
In freezing weather	Cuts 2″–6″	Cuts 2″–4″	Cuts 1″–4″	Cuts ½″–1″	Cuts ½″–1″	Cuts ½″–1″	....	Cuts ½″	Cuts ½″	Cuts ½″	Cuts 6″–10″
Frequency of re-shaping: Times per year	10–20	10–20	15–25	None in 4 yrs.	None in 3 yrs.	None in 4 yrs.	None in 8 mos.	None in 3 yrs.	None in 7 mos.	None in 3 yrs.	2–4
Sand No. 10, percentage	12.5	....	....	6.3	....	34.0	....	7.2	1.5	10.0	....
Sands Nos. 20–60, percentage	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2
Total sand, percentage	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Total clay, percentage	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3

NOTES :—* Data for No. 223, based on roads built near Augusta, Ga.
No. 272, Road in use 8 months at time of report.
No. 416, Road in use 7 months at time of report.
No. 282, practically an earth road.

Considering only the second and third groups, the sand-clay analyses may be condensed into Table 4.

TABLE 4.—Comparative Analyses of Sand-Clay Groups.

	Inferior Sand-Clays.		Superior Sand-Clays.	
	Limits.	Average.	Limits.	Average.
Sand total	52.6–60.0%	56.3%	63.8–71.5%	67.6%
Sand Nos. 20-60	32.3–41.0%	36.6%	45.0–57.9%	51.4%
Sand No. 100 and dust	10.0–17.0%	13.5%	4.1–13.8%	8.9%
Clay total	40.0–47.4%	43.7%	28.5–37.0%	32.7%

The principal differences between the two groups are:

1. The total sand content in inferior mixtures averages about 11% less than in superior mixtures;
2. The total quantity of sand from Nos. 20 to 60, in the former averages 15% less than in the latter.

Effect of Size of Sand.—Although the quantity of clay is nearly the same in Nos. 106 and 282, the former is a much more durable material. Roads built of the former withstand severe rains far better than those built of the latter material. To account for this, it must be remembered that the analysis of the whole sample of No. 106 is: Sand No. 10, 6.5%; sand No. 20 to dust, 43.7%; clay 43.8 per cent. The difference in the clay content is only about 4% in the two samples, which seems too little to account for the great difference in their conduct. By reference to the sand analyses of the two samples, it is seen that No. 106 contains nearly three times as much of the No. 20 and No. 40 sands, nearly the same quantities of Nos. 60, 80, and 100 sand and only one-third as much dust, as No. 282. This considerable difference in the ratio of coarse to fine sands explains, in large measure, the difference in the durabilities of the two materials.

Conclusions.

A study of nearly a thousand analyses, in the laboratory, and a comparison of the results attained with such materials in actual road construction, have led the writer to formulate the following tentative working rules for examining sand-clay mixtures and as a basis for making up proper combinations for artificial mixtures:

(1) The total relative sand content, disregarding the size of the sand grains, is no criterion of the value of the material.

(2) The sand smaller than No. 60 is of little value in the mixture, that smaller than No. 100, except in very small quantities, is detrimental.

(3) The greater the proportion of coarse to fine sand the harder and more durable will the road surface be.

(4) For the best possible results with sand-clay mixtures, the sand smaller than No. 10 and larger than No. 60 should not be less than 45% nor more than 60%, by dry weight, of the entire sample. In addition, the sand smaller than No. 10 and larger than No. 60 should be composed of about equal parts of Nos. 20, 40, and 60. The total sand content should in no case exceed 70% by weight, of the total sample.

(5) Test cylinders of the sand-clay mixture, 1 in. in diameter and 3 in. long, should, when thoroughly dried in air bath at 100° cent., take at least 2 min., when immersed in water at 21° cent., to crumble down to the natural slope of the material, and preferably should take 6 min. If the cylinder fails in this test, it should be regarded with suspicion. If the sand analysis is poor and the cylinder test is also poor, the material is not worth using.

(6) Test cylinders, made from the clay removed from the sample, 1 in. in diameter and 3 in. long, should take at least 2 min. to crumble down to the natural slope of the material, when immersed in water at 21° cent. If it fails in this test, but passes the test of the preceding paragraph, it may be used, but it indicates a poor quality of binder.

Approximate Field Method for Examining Sand-Clay Mixtures.

As a rapid aid in forming a judgment in the field as to the value of any mixture, the writer offers the following: A buggy inspection is made of the natural soil on both sides of the road to be improved. Samples are taken wherever the surface appearance seems to indicate favorable material. These samples are placed in paper bags, with notes of the location, etc. The samples are usually taken for a depth of 4 in. Each sample can be dried sufficiently by spreading out in the sun in a thin layer. When dried, the sample may be placed in

a No. 10 sieve and screened, and note made of the relative weights of the residue on the screen and the finer material. Then 100 grammes of the part passing this screen are weighed and placed in an evaporating dish. By careful washing, in a few minutes practically all the clay may be washed out, and is rejected. The sand residue is then dried with an alcohol flame, and weighed. It is then placed in a No. 20 sieve nesting over a No. 60 sieve. By screening the sand thus it can be quickly separated into three sizes: No. 20, between Nos. 20 and 60, and smaller than No. 60. These three portions can be weighed, and from this information a very good idea may be obtained of the comparative value of the material, without additional tests.

The proportion of the material coarser than No. 10 indicates the proportion of gravel present. The separation into No. 20 size and between Nos. 20 and 60 shows practically into what class the mixture would fall, and the portion smaller than No. 60 would indicate the quantity of almost worthless material in the mixture. By using Fig. 1 a very good idea of the quality of the sample would be gained.

Methods of Sand-Clay Construction.

There are two general methods of construction in use, one in which the natural mixture of sand and clay is used, the other in which an artificial mixture is made by using two or more materials which are mixed by plowing together, puddling, etc.

Construction with Natural Mixtures.—The method which the writer believes is simplest, and produces excellent results, is as follows: Natural mixtures of the proper proportions of sand and clay may be found as the natural topsoil of cultivated farms, or may be found below the surface at various depths. The material comprising the topsoil of cultivated land, when composed of the proper combination of sand and clay, has probably been more thoroughly weathered and therefore is less likely to wash and disintegrate when placed on the road. Cultivation has also produced a more thorough and complete mixture of the two materials.

In the northern half of Georgia the topsoil in large areas, especially on the tops of ridges, for a depth of from 6 to 12 in., is found to be an excellent sand-clay mixture. In the southern half of Georgia, a natural mixture of sand and clay is often found at a depth of from 2 to 5 ft. below the surface. Such material is not usually very well

JEFFERSON ROAD, CLARKE COUNTY, GEORGIA.

FIG. 7.—TOPSOIL PLACED SIX HOURS BEFORE PHOTOGRAPH WAS TAKEN.

FIG. 8.—TOPSOIL AFTER ONE WEEK OF TRAFFIC, WITH THREE HEAVY RAINS.

FIG. 9.—JEFFERSON ROAD AFTER SIX MONTHS OF TRAFFIC, OCTOBER, 1912, TO MARCH, 1913. TWELVE HOURS AFTER HEAVY RAIN.

weathered, so that, after being placed in the road, a certain quantity is found to wash out quite readily with the first few rains.

In either case the method of construction may be the same. The sub-grade of the roadway is brought to a level or slightly convex cross-section. The sand-clay is then placed in a continuous layer, from 10 to 12 in. thick, the material being spread as fast as delivered and not dumped in piles here and there. This layer is spread for a width of 20 ft. for a nominal 30-ft. roadway. After a sufficient quantity has been placed in this manner, an ordinary road machine is drawn along the ditch line, cutting about 4 in. deep at the outside, and the blade is set so as to cast the material from the ditch against the edge of the sand-clay layer. In this way a shoulder is built up against the sand-clay to hold it in place. This also shapes the ditch. After both sides have been thus shaped, the road machine, in successive passages, rounds up the cross-section of the sand-clay so as to give proper crown to the roadway and a smooth line from the crown to the ditches. As soon as the road is shaped, traffic and the construction teams begin to compact it, and it rapidly becomes consolidated without the use of a road roller. As the consolidation progresses, ruts are formed, and they should be filled and a proper cross-section maintained by the occasional use of the road machine for a period of about 2 months. Unless this is done, the road surface will become rutted and rough, and eventually compacted with a concave crown which will prevent proper drainage. After the material has been consolidated into a hard mass, the difficulty of securing a good cross-section is largely increased.

The cross-section which seems to have given the most generally satisfactory results is a parabolic form with a crown of ½ in. per ft., that is, for a roadway surfaced for a width of 20 ft., the crown would be 5 in., and the height of the center of the road above the ditch (for a road having a width of 30 ft. between ditches) would be 7.5 in. With steeper crowns than this it has been found that the surface cuts into a series of parallel ridges running from the wheel tracks to the ditches and making it very disagreeable for travel. If less crown is given, the provision for wear is too small, and the drainage may not prove satisfactory after a comparatively short time.

For several months rains are apt to soften the top crust and cut up the smooth surface, but if patience is exercised and the road

machine is used to maintain the cross-section properly, it will be found that the puddling action of the traffic when the road softens is a great aid to final consolidation.

Construction with Artificial Mixtures.—From analyses made of materials proposed for use on account of accessibility, and from a study of their sand analyses, the proper ratio in which two or more materials should be mixed can be determined so as to secure the best possible results with the available materials. Three cases arise in which artificial mixtures are to be used:

(1) Sand foundation, where clay is to be hauled and proper mixture made by disk plowing and puddling.

(2) Clay foundation, where sand is to be hauled and mixture made as above.

(3) Soil foundation, where both sand and clay are to be hauled and mixture made.

In any of these three cases the proper mixture of the materials and the puddling action of traffic are necessary to secure a good consolidation. It takes considerable labor to secure a satisfactory mixture, but, except for this, there is no essential difference in the fundamental principles applying to construction with either artificial or natural sand-clay mixtures. The use of the road machine to maintain the cross-section and the height of the crown should be the same for each type of construction. For the softer varieties of sand-clay, the split-log and other forms of light drags may be used effectively in maintenance.

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REPORT ON A SERIES OF TESTS ON CONCRETE COLUMNS REINFORCED WITH A SPIRAL OF STEEL.

By C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie,* and C. O. Carey.*

To be Presented March 18th, 1914.

General Statement.

In 1907-08 the writers were designing some structures in which certain columns were required to carry heavy loads and would be subjected to considerable shock. The nature of the structures was such that columns of reinforced concrete offered certain advantages, sufficient in importance to warrant the use of that material, even though low stresses and consequently an excessive quantity of material might be required to insure safety. At about this time a number of failures of reinforced columns in important structures at various places in the United States were reported in the engineering journals, sounding a warning against a too liberal estimate of their strength, and plainly indicating the need of a careful study of such members from the experimental standpoint as a basis for safe design. Some experiments then made on the ultimate strength of columns with longitudinal steel

* Messrs. Brodie and Carey are Instructors in the University of Michigan.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

bars embedded in the concrete indicated that such reinforcement, unless hooped or banded at frequent intervals, could not be relied on to increase the strength of the column, and in some cases might even render its strength less. These experiments, however, were few in number, and the time available did not permit of their being carried out with sufficient care to render the results thoroughly trustworthy; but they served to turn attention to the hooped column, which had been discussed by Considère in his treatise, regarding which experimental information was very meager.

A series of tests on small columns was planned and partly carried out during the first half of 1908, but the departure of Mr. Wrentmore for the Philippines, in July of that year, interrupted the work, and since that time it has been continued by Messrs. Brodie and Carey at irregular intervals, at such times as their University duties would permit.

In all, 115 columns have been made, and usable results have been obtained from 104 of these; the remaining 11 were broken in storage or in handling, were found faulty in construction, or showed such erratic results under tests as to warrant their being thrown out of the list. Those which will be discussed here are Nos. 10 to 22, and 36 to 115, both inclusive, except Nos. 40, 55, 83, 86, 88, 91, 103, 108, 109, and 112, a total of 83 columns. The other 21, making up the 104, include the columns reinforced with longitudinal rods previously referred to, and some special cases which are not germane to the present discussion. The series is still incomplete, some of the sub-groups showing no tests, or only one or two, and others showing wide variations and requiring more tests to determine a dependable average. Moreover, the extensometer tests on a large percentage of the columns were only carried to 700 or 800 lb., and, later, when a comparison of results was made, it was found that it would have been desirable to have carried all tests to at least 1 400 lb. per sq. in. Some columns gave usable results on only one part of the test, as on ultimate strength alone, or extensometer measurement alone; however, in view of the care which has been exercised in carrying out the work, it is believed that the results obtained are worth placing on record, and they are here presented in the hope that they may add in some measure to the sum total of useful information.

Acknowledgment.

The writers take pleasure in acknowledging the assistance received from the Dean of the Department of Engineering of the University of Michigan in granting the use of the University laboratories and equipment for carrying on these tests, to Gardner S. Williams, M. Am. Soc. C. E., for suggestions which were of assistance in securing accuracy and uniformity in the results, and to H. H. Atwell, Assoc. Am. Soc. C. E., and Messrs. J. Schmutz and L. H. Neilson, of the Faculty of Surveying, and Messrs. H. A. Shuptrine, E. Olmstead, L. R. Manville, and M. D. Bensley, students in Engineering, all of the University of Michigan, who gave their time and work to assist in carrying on the tests.

Aim of the Tests.

The tests were planned to determine the longitudinal and lateral deformations of the columns under various loads, increasing from zero to something above the probable maximum working stress, with varying quantities of reinforcement and at different ages, also the ultimate strength and the load at the first visible sign of failure. From the deformation could be determined the change in unit length for various loads, the modulus of elasticity, and Poisson's ratio. Comparison of the results should show the influence of the reinforcement on these several quantities.

Description of the Columns.

All columns were approximately 4 in. in diameter and 27 in. long, over all, cast in a galvanized-iron mould set on a wooden bottom. The mixture throughout was the same, 1 : 2 : 4 by loose volume. The cement was Peninsular brand, bought from a dealer from stock, in fair condition, and as ordinarily used in construction. It was sampled and tested as for construction work, passing satisfactorily the standard tests recommended by the Special Committee (of the American Society of Civil Engineers) on Uniform Tests of Cement. The sand was bank sand from a drift deposit at Ann Arbor, secured from a contractor who made a business of furnishing it to the building trades, and was of a grade acceptable for first-class work. The granulometric composition is shown by Table 2. The coarse aggregate was broken limestone from the quarries at Monroe, Mich., and was passed through a $\frac{3}{4}$-in. and retained on a $\frac{1}{4}$-in. screen.

The mixing was done by hand, and under the personal supervision of the writers. In order to fill well about the spirals, it was necessary to use a wet mixture; it was too wet to be tamped with a small rod, but not so wet as to cause the materials to separate in pouring.

The reinforcement in Columns 10 to 14 was furnished by a commercial firm, built up and ready to be placed in the forms. The spiral was of high-carbon steel, $\frac{1}{8}$ in. in diameter, spaced as shown in Table 3. The spacing members were four $\frac{1}{8}$ by $\frac{1}{16}$-in. steel bars. No tests were made to determine the physical constants of this reinforcing material. These columns were tested only for ultimate strength and first sign of failure, and, for purposes of comparison, are included with similar columns tested at other laboratories.

The reinforcement in Columns 15 to 22 was wound on the concrete shaft, after casting, by placing the column in a lathe and winding the wire on under constant tension—sufficient to bed it slightly in the concrete. This was done just before testing, at the age of 28 days. The end of the wire was anchored securely to the column very near one end, the tension was applied, and a dozen turns were wound on as close as they would lie in order to insure against failure at the end; the spiral was then spaced by using the lead screw until within 1 in. of the other end; then another dozen turns were run on close together. The wire was then brought under the anchor and fastened, the tension was released, and the wire cut free. On these columns the wire had a diameter of 0.079 in., an area of 0.0049 sq. in., and the tension, as given by a spring balance, was $62\frac{1}{2}$ lb. on some columns and 125 lb. on others, corresponding to unit stresses of 12 750 and 25 500 lb. per sq. in.

In the remaining columns in which reinforcement was used the spirals were made up and set in place in the form, and the concrete was then poured. The longitudinal members of the reinforcement were barely heavy enough to hold the spiral in place; they were strips of No. 24 gauge galvanized iron, $\frac{7}{16}$ in. wide, with notches, $\frac{7}{32}$ in. deep, accurately spaced by machine; the consecutive turns of the spiral were dropped into these notches and fastened by pinching down the lugs formed in cutting the notches. Four such spacing strips were used for each coil, and, as they were so light, considerable care was necessary to insure the proper location of the reinforcement. On final test to destruction, however, all spirals were found to have been spaced and

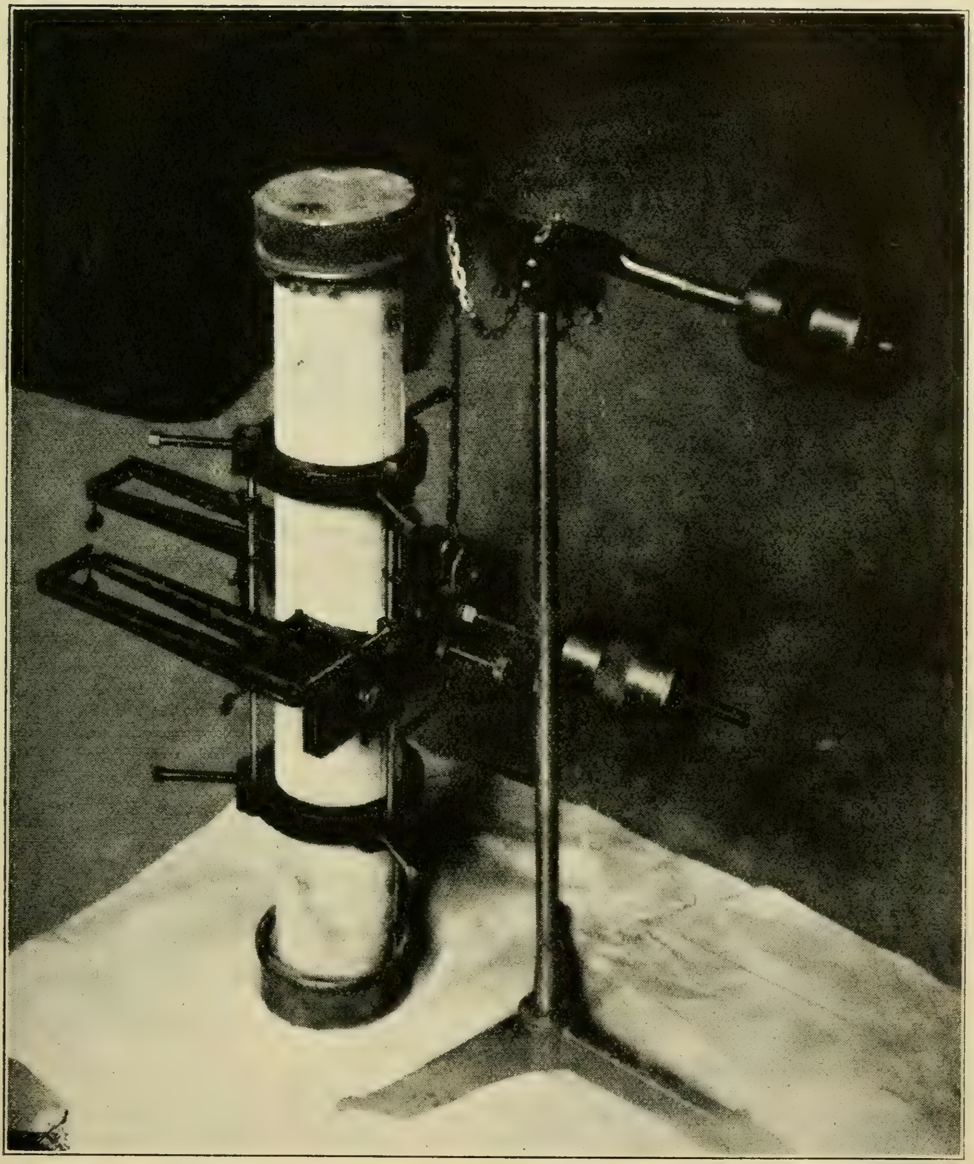

FIG. 1.—EXTENSOMETER.

centered accurately, although in some cases the longitudinal members had taken a spiral position, as may be seen on Fig. 3. A number of extra turns were given at each end of the coil, to make sure that failure should occur in the middle of the shaft, and a circular piece of ¼-in. mesh hardware cloth was placed in each end of the column to give it additional strength. In spite of these precautions, however, a number failed by splitting or crushing at the end, thus failing to develop the full strength of the shaft. The longitudinal spacing strips were not taken into account in estimating the percentage of reinforcement, as they were considered to be too slender to affect sensibly the column under test.

After being cast the columns stood for 24 hours in the forms, and then, except Nos. 10 to 22, were placed in water in the large naval experimental tank, where they remained until removed for testing. Nos. 10 to 22 were removed from the forms at 24 hours and cured in air in the laboratory.

The wire used in the spirals was of unannealed, copper-washed, soft steel, bought from a hardware dealer from stock. Under test it showed an average yield point, ultimate strength, and reduction of area as given in Table 1.

TABLE 1.

Diameter, in inches.	Ultimate strength.	Yield point.	Reduction of area. Percentage.	No. of tests.
0.105	92 330	85 780	50	35
0.079	95 260	87 870	50	45
0.062	78 000	70 000	80	45

The fracture was deeply cupped in all cases, and on crushing the columns the wire invariably broke with the same appearance of reduced area and cup.

The Instruments.

The instruments used were a 200 000-lb. Riehlé testing machine and an extensometer. A 100 000-lb. Olsen machine was available, but the larger clearance, the greater freedom from vibration, and the fact that it was not so frequently in use made the larger machine more desirable. The column was placed in the machine on a cushion of sand in a cast-iron cup which had a machined surface for bearing on the bed of the testing machine.

The extensometer, Fig. 1, was designed by the writers and built in the Instrument maker's shop of the University. The small variation to be measured rendered necessary extreme care and accuracy in construction, and especially the elimination of all lost motion. Knife-edges were ground straight in a Universal grinding machine after tempering, and the V-notches for the knife-edge bearings were made in two parts and similarly ground to true lines. The five rollers of tempered steel were ground to true cylinders, and as nearly of the same size as practicable, and afterward Nos. 2, 3, 4, and 5 were calibrated against No. 1 in a specially designed device.

The small amount of lost motion is shown by the fact that when the column was loaded and the scale-beam rose and fell between its stops a distance of about $\frac{3}{4}$ in. at the free end, the lateral mirrors traveled to and fro on the scale, showing clearly the increase and decrease of diameter of the column due to the small variation of load produced by this motion of the scale-beam.

The construction of the instrument is shown by Figs. 1 and 2. For longitudinal measurement, there are two cast-iron rings, *q*, connected by three telescoping rods, each made up of the rods, *r* and *s*, and the tube, *t*. The rings, *q*, are fastened to the column by setting up the screws, *w*, to a firm bearing. The rods, *r* and *s*, are then set in their seats in the rings and the nuts lightly screwed down on the spherical heads, as shown in the small sectional view. The tube, *t*, is slipped on and firmly clamped to the short rod, *r*, the longer rod, *s*, being left free to slide within *t* as the column shortens under load. The bar, *u*, attached to *t*, holds the roller between its face and the bar, *s*, being held to a steady pressure by the adjustable spring, *v*. The average of the readings of the three rods gives the deformation of the column.

For lateral measurement, the frame, *a*, with its arms attached, is hung by a chain from the counterbalance arm shown in Fig. 1, by the hook at *g*, which is fast to the sliding bar, *e*, which is moved by *h*. The bar, *e*, and the counterweight on *d* are adjusted until the bar, *a*, hangs level, and the weight on the counterbalance arm is adjusted until it exactly balances the weight of the instrument. It is then adjusted to the column in place in the testing machine, with the knife-edges, *o*, and the screws, *b*, bearing against the small blocks of plate glass cemented to the column. The pieces, *j* and *k*, are bolted rigidly to *a* when in use. The arm, *m*, is held to a firm bearing on its

knife-edges, *n* and *o*, by a spring and plunger in *p*. The distance between these knife-edges is 1 in. and the length from *n* to the end of *m*, on which the roller bears, is 10 in. The roller is placed between the end of *m* and the bar, *l*, the latter being held by a spring to a firm

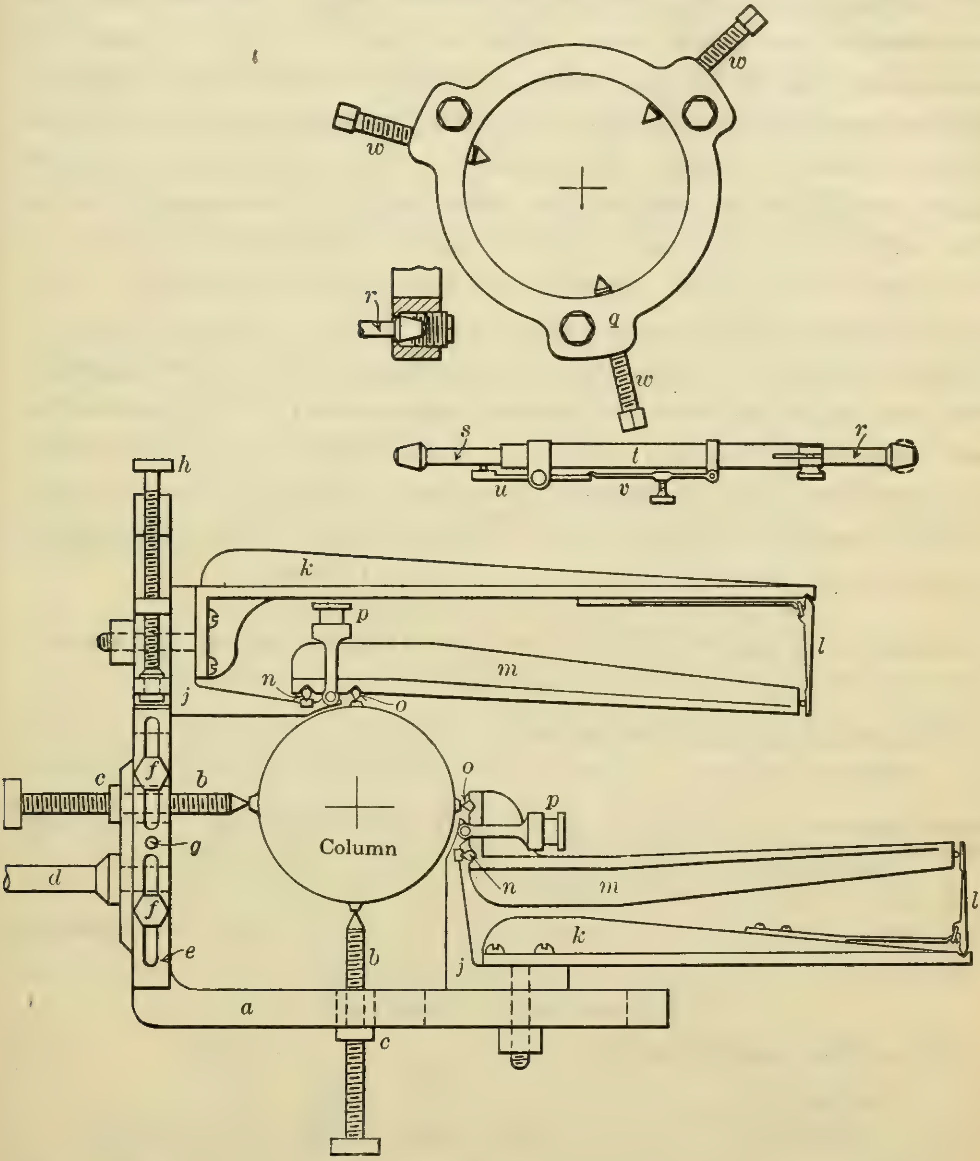

FIG. 2.

bearing against both its knife-edge seat on *k* and the roller. When all is adjusted, the screws, *b*, are made fast by the lock-nuts, *o*, so that all is rigidly connected to *a* except the arms, *m*, and the rollers. Any increase or decrease of diameter will cause *m* to rotate about *n*

and produce rotation of the roller. The average of the change shown on the two diameters is taken as the change in diameter of the column. The instrument when adjusted is free to move with the column, but none of its weight is carried by its bearing on the glass plates; it is held from slipping on these by the friction only. As the frame has considerable mass, any strong vibration tends to cause slipping and consequent loss of the experiment. It was found necessary, therefore, to disconnect the driving motor from the testing machine and run the machine slowly by hand. Each roller carries a small mirror, with its plane parallel to the axis of the roller, and the instrument is set so that all five mirrors come within the field of a telescope, and each can be placed on one of the cross-wires. The scales are then set at their exact distances from the several mirrors, and the readings of all five mirrors are taken at once, without shifting any of the recording parts, thus accomplishing the simultaneous measurement of the longitudinal and lateral deformations. When the extensometer tests of the column are completed, the extensometer is removed and the column is tested for the first sign of failure and ultimate strength. The complete test of one column required three men for about 4 hours.

TABLE 2.—Percentage of Voids and Granulometric Composition of Sand.

Weight of sample, 1 000 grammes. Voids by volume, 32 per cent.

									Passed.
Meshes, per inch......	4	6	16	20	30	74	100	200	200
Residue, grammes....	5	26	168	102	225	429	26	12	5
" percentage.	0.5	2.6	16.8	10.2	22.5	42.9	2.6	1.2	0.5

Total.. 99.8 per cent.

Extensometer Constants.

The roller coefficients were:

Roller No........	1	2	3	4	5
Coefficient........	1.000	0.994	0.986	0.965	0.987

Each reading from a roller is multiplied by its coefficient to give the corrected reading.

Referring to Fig. 2: From n to the end of m is 10 times the length n-o. The roller between m and l has a diameter, $2r = \frac{1}{10}$ in., approximately. R is the distance from the center of the roller to the scale.

taken as 100 in. for the transverse extensometer and 50 in. for the longitudinal one. s_1 is the distance traversed on the scale by the lateral mirror when a load is applied, corrected as above stated. s_2 is the same for the longitudinal mirror. λ_1 is the unit lateral deformation for any load. λ_2 is the unit longitudinal deformation for any load.

If the roller turns through a small angle, θ, the distance passed over on the scale is $2R\theta$. At the same time, the motion of the end of m past l is $2r\theta$.

$$\text{Motion of end of } m : s_1 :: r : R :: \frac{1}{20} : 100 :: 1 : 2\,000.$$

Motion of end of m is 10 times that of o.

Therefore, motion of $o : s_1 :: 1 : 20\,000$. Hence the unit on the scale used being $\frac{1}{50}$ in., the motion of o necessary to make a reading of this unit is $\frac{1}{1\,000\,000}$, and the change of diameter of the specimen having a diameter of D is the scale reading divided by 1 000 000. Thus the change per inch of diameter is:

$$\text{Unit lateral deformation} = \frac{s_1}{1\,000\,000\,D} = \lambda_1.$$

For longitudinal deformations, the conditions are the same, except that no multiplying lever is used, and the value of R is 50 in. Therefore the change per unit length is

$$\text{Unit longitudinal deformation} = \frac{s_2}{50\,000\,L} = \lambda_2,$$

where L is the distance between the points of attachment of the rings to the column, taken as 12 in. for these tests. Then Poisson's ratio is

$$m = \frac{\lambda_1}{\lambda_2} = \frac{1\,000\,000\,s_2\,D}{600\,000\,s_1} = \frac{5\,s_2\,D}{3\,s_1}.$$

Procedure in Testing.

The column was taken from the tank and allowed to dry. Diameters were calipered in two directions at right angles to each other at the bottom, center, and top. Two circles with planes normal to the axis were struck, 12 in. apart, to locate the attachment of the rings for the longitudinal measurement. A third was similarly struck for the position of the points for the lateral measurement. On this last circle, the extremities of two diameters at right angles to each

other being located, small blocks of plate glass were cemented to the column with faces made parallel by pressing them to their bed between two parallel steel faces. When these were firmly set, the rings were attached and the column was set with top down on the bed of sand in the cast-iron cup. Cotton waste was tamped lightly around the column to retain the sand. The column, with cap in place, was then set in the testing machine in an upright position, the bottom end on a bed of sand in a cup like that used at the top, and a light load applied to hold it in place, care being taken to secure a full even bearing on the sand. The transverse extensometer was set and adjusted as heretofore described.

The zero reading was taken with a total load of 100 lb. on the column, and thereafter increasing loads were applied and readings taken at each load, up to a maximum, then decreasing loads down to zero were read. On a few columns the loads were run up and back a second time, but usually only one run was made. These readings being completed, the extensometer was removed and the column was tested to destruction.

Behavior Under Test.

The driving gears of the testing machine caused considerable vibration when in action, and in some cases caused the lateral extensometer to slip on the glass plates. The readings showed plainly when this took place, and several tests were necessarily discarded on this account. By running the machine slowly by hand, this trouble was minimized, and usable results were obtained. Usually, the readings were fairly uniform and consistent, but, occasionally, erratic results were found, such as one mirror starting off with negative readings for a few loads, and gradually returning to positive ones as the loads were increased. There was commonly a noticeable difference in the readings between the two lateral, or among the three longitudinal, mirrors, showing that the column underwent lateral deflection. However, when the readings were averaged and plotted, the results usually were very fair curves.

In applying the increasing loads, the weight was set at the proper mark and the load was run up until the beam rose, the reading then being taken. The beam was never held up by the column, but always dropped showing progressive deformation.

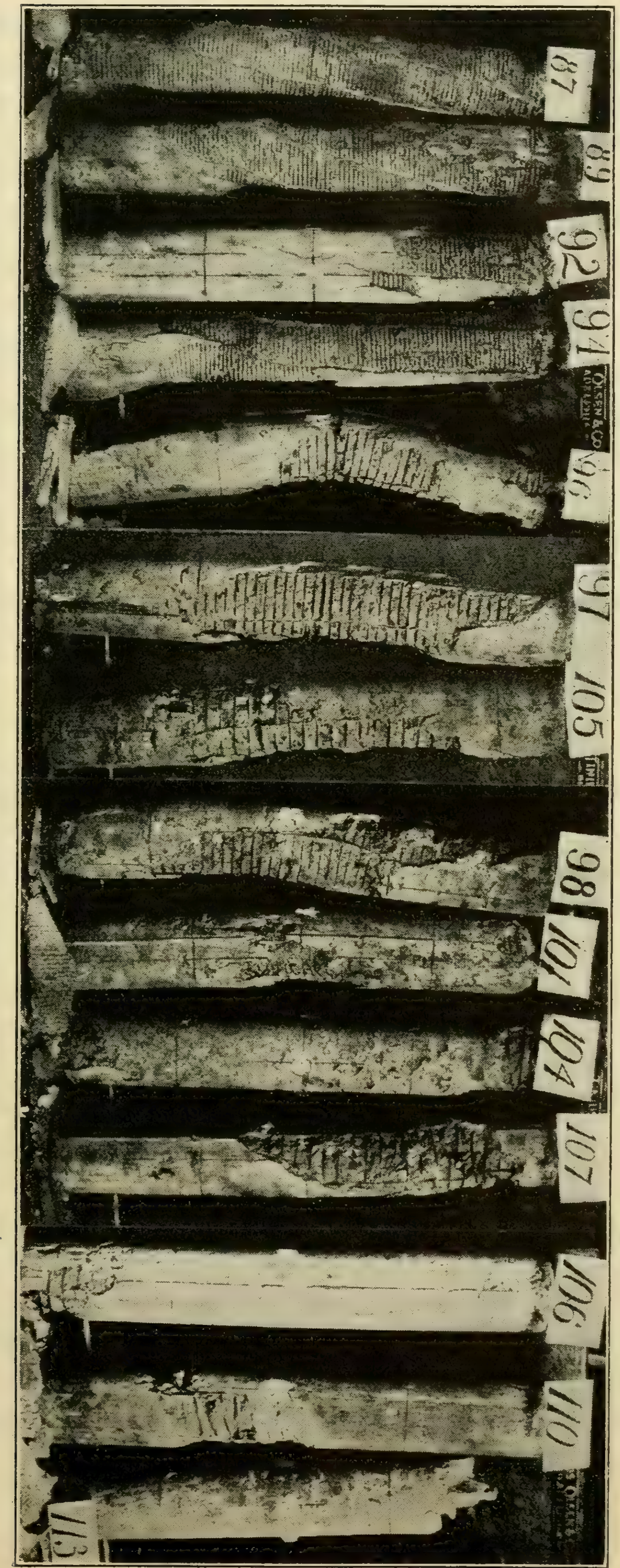

FIG. 3.—APPEARANCE OF CERTAIN TYPICAL COLUMNS AFTER TEST.

In loading to destruction, considerable lateral deflection usually appeared, the column frequently approximating the characteristic curve of the slender column with fixed ends. Extreme care was taken to observe the first sign of failure, which was shown by a flaking of the surface cement, commonly near the center of length of the column. In the columns without reinforcement, failure was abrupt, only two or three giving any previous indication, and that at a very small percentage below the ultimate load. In those with spiral reinforcement, it was always gradual, giving ample warning and time for observation. The outer concrete separated from the core in considerable sections and fell away, leaving the spiral exposed; with increasing loads, deflection appeared, the concrete of the core began to force out the hooping, and finally the latter broke, the concrete flying out at the point of the break as though blown out by a small charge of explosive; the wire showed a clean tension break. In a few cases the column failed at or near the end, showing imperfect construction. These cases were discarded unless the values were fairly high, as the local failure prevented the column from developing its full strength.

Computation of Values.

The area of cross-section was computed for all plain columns as a circle having a diameter equal to the mean of the six measured, two each at top, center, and bottom. The unit load for values of s_1, s_2, e, and m, for both plain and reinforced columns, are based on the area thus found.

For reinforced columns, the ultimate unit strength and the unit load at the first sign of failure are based on the area of core enclosed by the spiral. The percentage of steel is based on the volume of steel and the volume of core. The percentage shown in Column 14 of Table 3 is the ratio of total load at first sign of failure to total ultimate load. The average lateral deformation is the average of the distances traversed by the lateral extensometer mirrors on the scale, in fiftieths of an inch; and, as the column diameter on which this measurement was taken was in all cases very near 4 in., this reading gives the increase in lateral dimension of the unit cube in units of $\frac{1}{4\,000\,000}$ in. Similarly, the longitudinal deformation is the average of the readings of the three longitudinal mirrors, and gives the longitudinal deformation of the unit cube in units of $\frac{1}{600\,000}$ in. The

value of Poisson's ratio would be approximately $\frac{4\ 000\ 000\ s_2}{600\ 000\ s_1}$, or $\frac{20\ s_2}{3\ s_1}$, but, in computing this, the exact diameter was used instead of the approximation of 4 in., so that the coefficient varies slightly from $\frac{20}{3}$.

The value of $e = \frac{E \text{ of steel}}{E \text{ of concrete}}$, and is used because it is quite as readily computed and plotted as the E of concrete, and, for purposes of this discussion, is somewhat more useful. It is computed as follows:

E of steel is taken as 29 000 000;

f_c = stress on concrete, in pounds per square inch;

$$E_c = \frac{600\ 000\ f_c}{s_2};$$

$$\frac{E_s}{E_c} = \frac{29\ 000\ 000\ s_2}{600\ 000\ f_c} = \frac{145\ s_2}{3\ f_c}.$$

The 10-in. slide-rule was used wherever time could be saved thereby, except for values which could be taken directly from tables.

Figs. 4 to 9, inclusive, show typical curves of s_2, e, and m, plotted to uniform scale, for columns Nos. 38, 46, 51, 58, 61, and 63. The unit for the s_2 curves is $\frac{1}{600\ 000}$ in., that for e and m is unity. The curves for any column in the series may be plotted from the data for s_2, e_1, and m, in Tables 5, 6, and 7. The curves on Figs. 4 to 9, however, show the effect of unloading the column, as well as loading, but the tables do not.

Comparison of Ultimate Strength.

The effect of the reinforcement on the ultimate strength is shown by the comparisons in Table 4. The columns are grouped as follows:

Group	1.	Reinforcement	0.00 per cent.
	2.	"	0.44 to 0.54 per cent.
	3.	"	0.67 to 0.86 "
	4.	"	1.16 to 1.35 "
	5.	"	1.41 to 1.69 "
	6.	"	2.02 to 2.32 "
	7.	"	4.10 per cent. (One column only.)

TABLE 3.—LIST OF COLUMNS TESTED, AND RESULTS OF TESTS OF ULTIMATE STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Column No.	Average diameter.	Mean area.	Age, in days.	Diameter of coil.	Diameter of wire.	Pitch of coil.	Tension.	Percentage.	FIRST SIGN.		ULTIMATE.		Percentage.
									Total.	Unit.	Total.	Unit.	
10	4.03	12.76	28	3.6	0.105	0.50	......	1.87	18 000	1 770	30 770	3 020	58
11	4.05	12.88	28	3.5	0.104	0.75	......	1.25	23 000	2 390	30 100	3 130	76
12	4.03	12.76	28	3.5	0.134	0.75	......	2.09	23 000	2 390	32 300	3 360	71
13	4.03	12.76	28	3.5	0.165	1.00	......	2.30	26 000	2 700	42 100	4 370	62
14	4.06	12.95	28	3.5	0.165	1.50	......	1.54	20 000	2 080	32 500	3 380	62
15	4.02	12.69	28	4.02	0.079	0.25	62	1.95	32 000	2 520	52 800	5 490	60
16	4.19	13.79	28	4.19	0.079	0.25	62	1.87	35 000	2 540	45 550	4 740	77
17	4.20	13.85	28	4.20	0.079	0.25	125	1.87	37 000	2 670	51 100	5 310	72
18	4.18	13.72	28	4.18	0.079	0.33	125	1.26	38 000	2 760	47 670	4 950	80
19	4.05	12.88	28	4.05	0.079	0.50	62	0.97	34 000	2 640	39 600	4 120	86
20	4.06	12.95	28	4.06	0.079	0.50	125	0.96	27 700	2 140	34 200	3 560	81
21	4.17	13.66	28	4.17	0.079	0.33	125	0.41	34 000	2 490	49 000	5 090	69
22	4.03	12.76	28	4.03	0.079	0.70	125	0.69	37 000	2 900	40 950	4 250	90
36	4.03	12.76	42	......	......	......	......	......	14 120	1 108	......	......	100
37	4.05	12.88	41	......	......	......	......	......	12 260	950	......	......	100
38	4.02	12.69	63	......	......	......	......	......	22 660	1 790	......	......	100
39	4.05	12.88	40	......	......	......	......	......	14 560	1 130	......	......	100
41	4.05	12.88	40	......	......	......	......	......	10 500	815	11 640	905	90
42	4.03	12.76	58	......	......	......	......	......	21 980	1 720	......	......	100
43	4.07	13.01	73	......	......	......	......	......	16 600	1 275	......	......	100
44	4.05	12.88	74	......	......	......	......	......	17 840	1 385	......	......	100
45	4.03	12.76	268	......	......	......	......	......	34 980	2 740	......	......	100
46	4.03	12.76	296	......	......	......	......	......	37 700	2 955	......	......	100
47	4.05	12.88	288	......	......	......	......	......	38 000	2 955	......	......	100
48	4.04	12.82	414	......	......	......	......	......	45 800	3 570	......	......	100
49	4.02	12.69	71	......	......	......	......	......	29 890	2 345	......	......	100
50	4.05	12.88	65	......	......	......	......	......	......	......	......	......	......
50*a*	4.03	12.76	340	......	......	......	......	......	37 000	2 900	......	......	100
51	4.04	12.82	434	......	......	......	......	......	46 300	3 600	......	......	100
52	4.04	12.82	440	......	......	......	......	......	35 000	2 725	......	......	100
53	4.02	12.69	295	......	......	......	......	......	39 570	3 120	......	......	100
54	4.05	12.88	441	......	......	......	......	......	40 700	3 160	......	......	100
56	4.04	12.82	58	3.50	0.079	0.33	......	1.68	25 000	2 600	40 590	4 220	61
57	4.02	12.69	59	3.56	0.079	0.33	......	1.65	24 260	2 430	39 000	3 940	62
58	4.03	12.76	61	......	......	......	......	......	22 420	1 760	......	......	100
59	4.05	12.88	67	3.50	0.105	0.75	......	1.31	30 600	3 180	40 250	4 184	76
60	4.04	12.82	311	3.62	0.079	0.75	......	0.72	32 000	3 100	38 200	3 700	83
61	4.04	12.82	77	......	......	......	......	......	29 450	2 300	......	......	100
62	4.03	12.76	434	......	......	......	......	......	36 350	2 850	......	......	100
63	4.05	12.88	395	3.69	0.105	0.75	......	1.24	27 500	2 575	37 500	2 910	73
64	4.03	12.76	435	3.88	0.079	0.75	......	0.67	37 500	3 180	41 500	3 520	90
65	4.05	12.88	316	3.62	0.105	0.75	......	1.26	37 500	3 630	42 160	4 085	88
66	4.03	12.76	74	3.62	0.079	0.75	......	0.72	19 570	1 895	29 000	2 810	67
67	4.02	12.69	323	......	......	......	......	......	31 300	2 470	......	......	100
68	4.03	12.76	321	3.69	0.105	0.75	......	1.24	31 400	2 940	36 570	3 420	85
69	4.03	12.76	323	3.62	0.079	0.75	......	0.72	24 700	2 390	31 350	3 040	78
70	4.04	12.82	393	3.37	0.079	0.25	......	2.32	27 000	3 020	53 690	6 000	50
71	4.05	12.88	317	3.56	0.062	0.75	......	0.45	27 120	2 720	27 450	2 770	98
72	4.03	12.76	388	3.62	0.079	0.75	......	0.72	31 000	3 000	33 500	3 246	92
73	4.04	12.82	428	3.62	0.105	0.75	......	1.26	46 500	4 500	48 600	4 710	95
74	4.06	12.95	370	3.62	0.062	0.75	......	0.44	47 000	4 550	47 800	4 630	98
75	4.04	12.82	300	3.56	0.062	0.62	......	0.54	34 000	3 400	40 500	4 090	83
76	4.03	12.76	376	......	......	......	......	......	27 250	2 140	28 000	2 200	97
77	4.03	12.76	277	3.62	0.105	0.62	......	1.51	38 000	3 680	42 700	4 140	89
78	4.04	12.82	281	3.56	0.062	0.62	......	0.54	19 000	1 900	31 300	3 160	60
79	4.06	12.95	416	......	......	......	......	......	28 060	2 170	......	......	100
80	4.05	12.88	417	......	......	......	......	......	29 000	2 250	......	......	100
81	4.02	12.69	270	3.62	0.079	0.62	......	0.86	34 000	3 290	36 250	3 520	93
82	4.04	12.82	413	3.62	0.062	0.62	......	0.52	33 500	3 240	33 700	3 265	99
84	4.03	12.78	42	......	......	......	......	......	......	......	......	......	......

TABLE 3.—(*Continued.*)

(1) Column No.	(2) Average diameter.	(3) Mean area.	(4) Age, in days.	(5) Diameter of coil.	(6) Diameter of wire.	(7) Pitch of coil.	(8) Tension.	(9) Percentage.	(10) First Sign. Total.	(11) First Sign. Unit.	(12) Ultimate. Total.	(13) Ultimate. Unit.	(14) Percentage.
85	4.05	12.91	42	......	......	......	......	......	....	......	....	......	......
87	4.03	12.78	35	3.37	0.105	0.25	......	4.10	27 600	3 090	71 300	7 965	39
89	4.04	12.84	35	3.37	0.079	0.25	......	2.31	21 600	2 420	49 900	5 580	43
90	4.04	12.80	35	3.37	0.079	0.25	......	2.31	....	......	....	......	......
92	4.04	12.80	34	3.37	0.062	0.25	......	1.41	21 500	2 410	24 000	2 680	89
93	4.03	12.76	34	3.37	0.062	0.25	......	1.41	....	......	....	......	......
94	4.03	12.76	35	3.37.	0.062	0.25	......	1.41	26 500	2 970	35 270	3 940	75
95	4.03	12.78	35	3.37	0.105	0.50	......	2.02	....	......	....	......	......
96	4.03	12.74	35	3.37	0.105	0.50	......	2.02	22 600	2 530	42 700	4 770	53
97	4.03	12.76	56	3.37	0.105	0.50	......	2.02	33 300	3 730	51 580	5 760	64
98	4.04	12.81	31	3.37	0.079	0.50	......	1.16	19 140	2 140	31 500	3 520	61
99	4.03	12.77	31	3.37	0.079	0.50	......	1.16	....	......	....	......	......
100	4.04	12.80	31	3.37	0.079	0.50	......	1.16	....	......	....	......	......
101	4.03	12.74	29	3.37	0.062	0.50	......	0.71	16 000	1 790	22 000	2 460	73
102	4.04	12.80	29	3.37	0.062	0.50	......	0.71	....	......	....	......	......
104	4.04	12.79	35	3.37	0.105	0.75	......	1.35	17 000	1 900	24 500	2 740	69
105	4.05	12.88	49	3.37	0.105	0.75	......	1.35	27 000	3 020	40 000	4 470	67
106	4.04	12.80	44	3.37	0.105	0.75	......	1.35	32 600	3 650	34 800	3 890	93
107	4.03	12.74	35	3.37	0.079	0.75	......	0.78	22 500	2 520	28 000	3 130	80
110	4.03	12.75	35	3.37	0.062	0.75	......	0.47	21 000	2 350	23 700	2 650	89
111	4.04	12.78	35	3.37	0.062	0.75	......	0.47	....	......	....	......	......
113	4.04	12.79	35	......	......	......	......	......	17 200	1 350	....	......	100
114	4.04	12.80	35	......	......	......	......	......	....	......	....	......	......
115	4.04	12.80	35	......	......	......	......	......	....	......	....	......	......

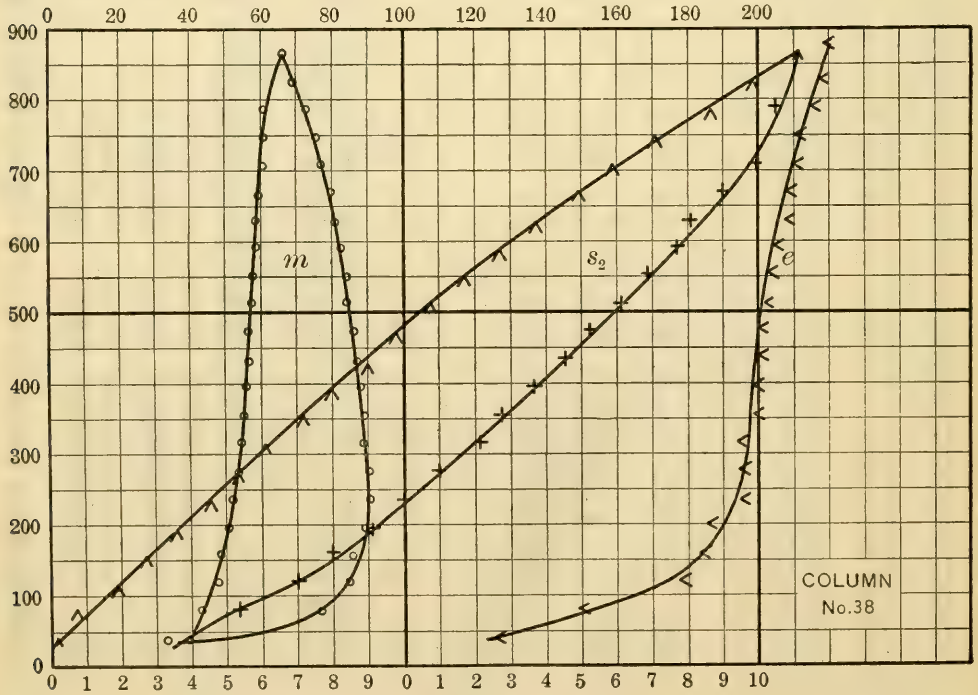

FIG. 4.

Plotting the average ultimate strength of each group, as shown on Fig. 10, gives four points for Group 1 and two for each of the others, except Group 7, which falls entirely off the diagram. The curve of Group 1 shows a consistent increase of strength with age, approximating in form to that of the age-strength curve of neat cement in tension, though the early increase is relatively less. For Groups 2 to 6 the ultimate strength is shown in full lines and the first sign of failure in broken lines.

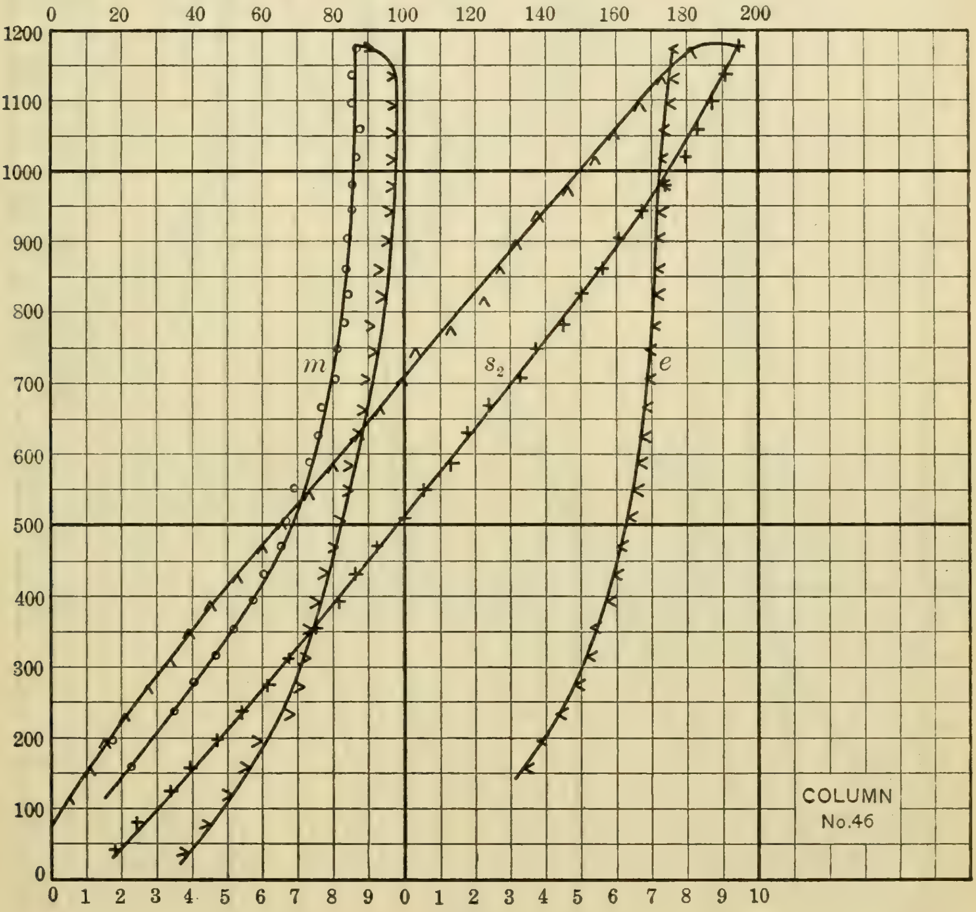

FIG. 5.

There are some irregularities in the relations of these lines, which are probably due to accidental causes, except the reversal of position of 2 and 3, which will be discussed later. However, all groups show an increase of strength with age, and generally with increasing quantities of reinforcement. The load for first sign of failure shows considerable irregularity in the long-time tests in Groups 5 and 6, but, as each of

these sub-groups contains results of one column only, such irregularity is plainly accidental.

Although the relation of the first sign of failure to ultimate strength is brought out more clearly by another diagram, this plotting is given to show the relations within the individual groups. The lack of har-

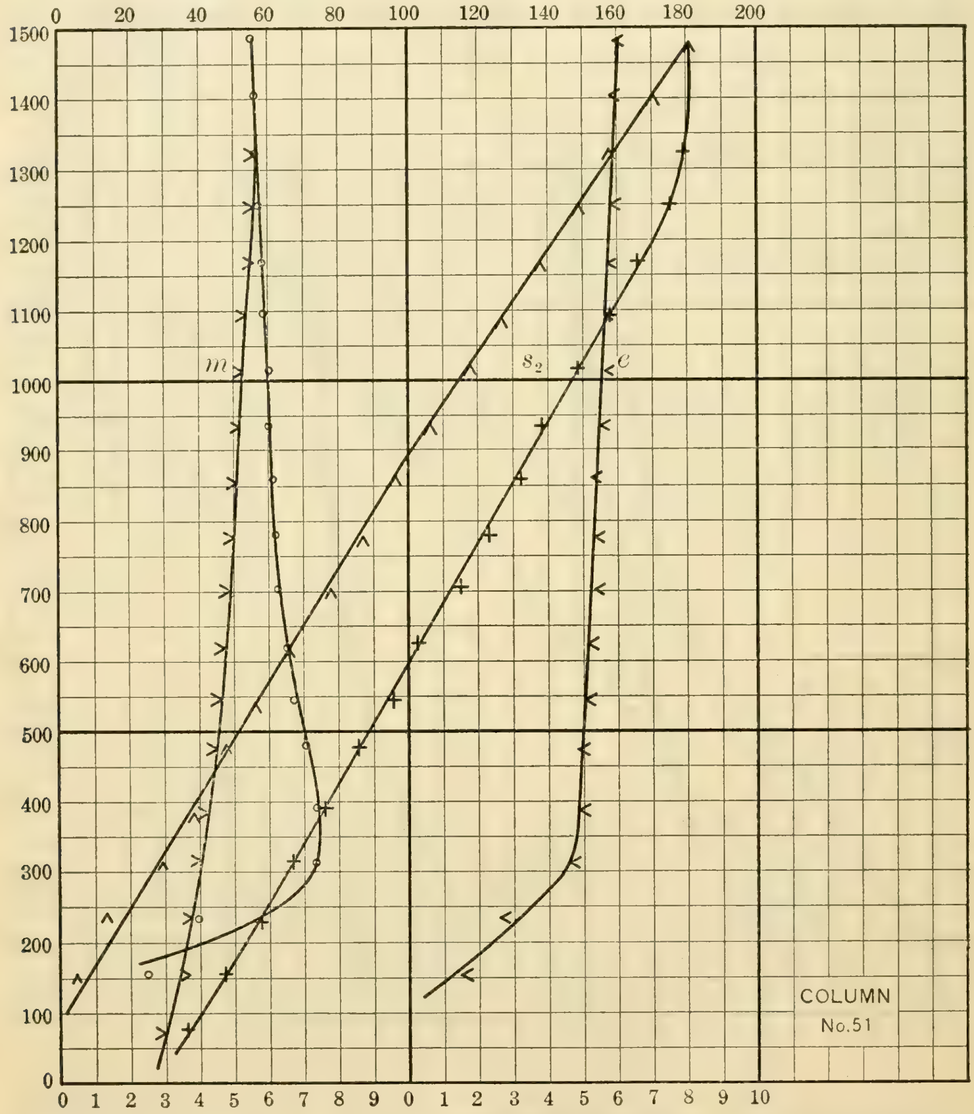

FIG. 6.

mony as the number of tests decreases, and the danger of generalizing from a few tests, are here clearly indicated.

The percentages of Column 14 of Table 3 are plotted on Fig. 11, using percentages of steel as abscissas and the values of Column 14 as ordinates. Here the short-time points, with the exception of that for

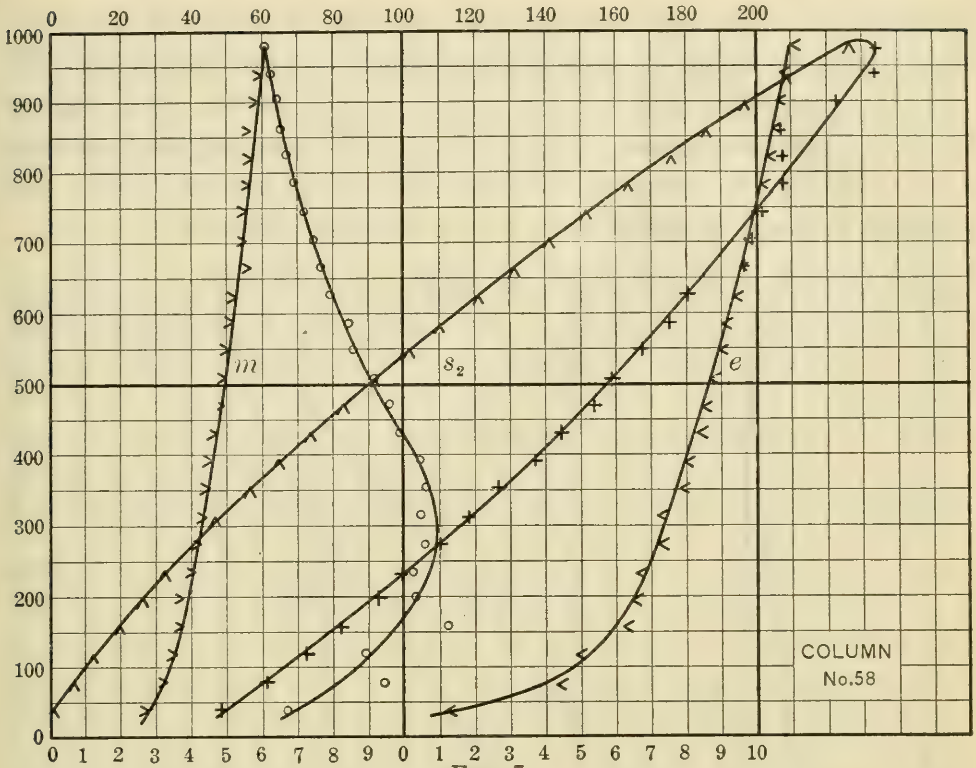

FIG. 7.

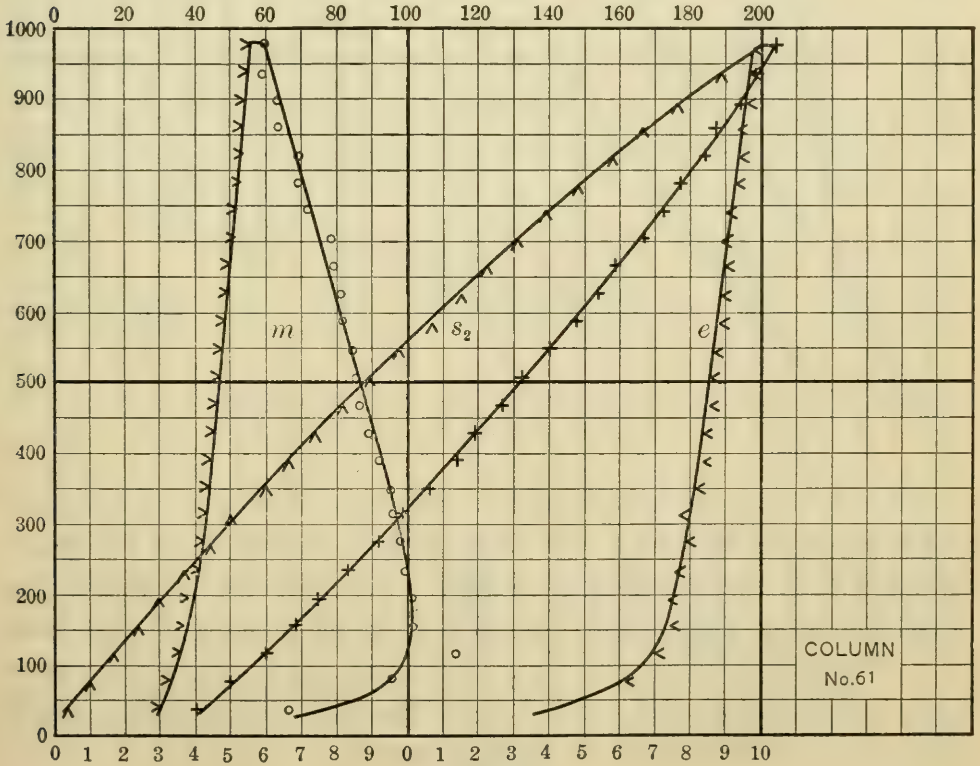

FIG. 8.

Group 3, lie close to a straight line through (0.100), and the long-time points lie on a curve at higher values. These lines show that, with from 0.80 to 1.60% of reinforcement, the columns at an age of 45 days, may be expected to give warning of approaching failure at about 80% of the ultimate strength, and, at an age of 1 year, this warning is given at about 90% of the ultimate strength.

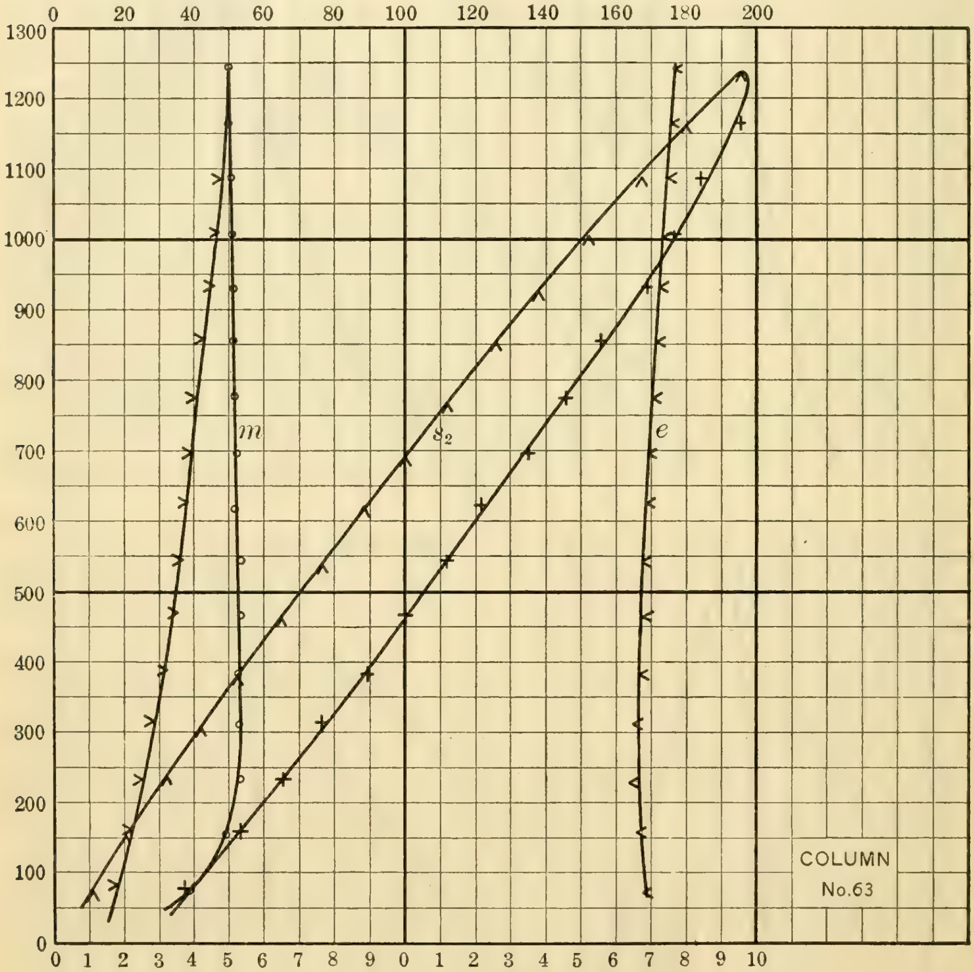

FIG. 9.

On Fig. 12 are plotted stresses at first sign of failure as ordinates with percentages of steel as abscissas, *M-N* being the curve for short-time and *O-P* that for long-time tests. For purposes of design, the values given by these curves would correspond roughly to the yield point of a steel member, in that, once determined with a reasonable degree of accuracy, this, rather than the ultimate strength, would control the allowable limit of working stress.

Although the small number of points leaves much to be desired, the general form of the curves seems to point to two conclusions: first, that the effect of hooping, in raising the first sign of failure, decreases with increasing age; and second, that even small quantities of steel exert an appreciable influence on this limit.

Plotting the group averages, with percentages of steel as abscissas and ultimate strength as ordinates, gives the curves of Fig. 13, *C-G-H* being the curve for an average age of 45 days, and *E-G′-K* that for approximately 1 year. These curves come nearly together at *G-G′*, and from that point lie close to the line, *A-B*, drawn from the origin through the point, *G*, the latter having the co-ordinates of approximately (0.0158, 4 000). Here the small number of tests in Groups 5 and 6, each three columns for short-time and one only for long-time, renders the portion, *G-H*, and especially, *G′-K*, questionable. In the absence of more complete data, assuming the curves as shown to be correct, within a reasonable limit of error, these curves show that, for percentages of steel greater than 0.0158, the ultimate strength of the column is not dependent in any measure on the age or strength of the concrete, but solely on the strength of the steel and the coefficient of friction of the aggregate, though with smaller quantities of steel the strength of the concrete is a factor, increasing in relative importance as the proportion of steel decreases. In other words, it appears that, with less than 0.0158 steel, when the cohesion of the concrete

TABLE 4.

GROUP 1.—DIVIDED INTO FOUR SUB-GROUPS, BASED ON THE AGE AT THE TIME OF TESTING.

(*a*)			(*b*)			(*c*)			(*d*)		
Column No.	Age.	Strength.	Column No.	Age.	Strength.	Column No.	Age.	Strength.	Column No.	Age.	Strength.
36	42	1 110	38	63	1 790	45	268	2 740	48	414	3 570
37	41	950	42	58	1 720	46	296	2 955	51	434	3 600
39	40	1 130	43	73	1 275	47	288	2 955	52	440	2 725
41	40	905	44	74	1 385	50*a*	340	2 900	54	441	3 160
113	35	1 350	49	71	2 345	53	295	3 120	62	434	2 850
			58	61	1 760	67	323	2 470	79	416	2 170
			61	77	2 300	76	376	2 200	80	417	2 250
Averages..	40	1 089	..	68	1 796	..	313	2 763	..	428	2 903

TABLE 4.—(*Continued*).

GROUPS 2 TO 6 ARE EACH DIVIDED INTO TWO SUB-GROUPS.

Column No.	Age.	Ultimate strength.	Percentage of steel.	First sign.	Percentage of ultimate.	Column No.	Age.	Ultimate strength.	Percentage of steel.	First sign.	Percentage of ultimate.
GROUP 2.											
110	35	2 650	0.47	2 380	89	71	317	2 773	0.45	2 720	99
						74	370	4 632	0.44	4 550	98
						75	300	4 091	0.54	3 410	84
						78	281	3 160	0.54	1 910	61
						82	413	3 265	0.52	3 240	99
Averages..	..	....	....	....	..	..	336	3 584	0.50	3 166	88
GROUP 3.											
66	74	2 810	0.72	1 890	67	60	311	3 702	0.72	3 100	84
101	29	2 460	0.71	1 790	73	64	435	3 520	0.67	3 170	90
107	35	3 130	0.78	2 520	80	69	323	3 040	0.72	2 390	70
						72	388	3 246	0.72	3 000	93
						81	270	3 520	0.86	3 300	94
Averages..	46	2 800	0.74	2 067	72	..	345	3 406	0.74	2 992	88
GROUP 4.											
59	67	4 184	1.31	3 180	76	63	395	3 520	1.24	2 570	73
98	31	3 520	1.16	2 130	61	65	316	4 085	1.26	3 640	89
104	35	2 740	1.35	1 900	69	68	321	3 420	1.24	2 940	86
105	49	4 470	1.35	3 020	67	73	428	4 710	1.26	4 500	96
106	44	3 890	1.35	3 640	93						
Averages..	45	3 761	1.30	2 774	73	..	365	3 934	1.25	3 412	86
GROUP 5.											
56	58	4 219	1.68	2 600	62	77	277	4 138	1.51	3 680	89
57	59	3 939	1.65	2 430	62						
94	35	3 940	1.41	2 960	75						
Averages..	51	4 033	1.58	2 663	66						
GROUP 6.											
89	35	5 580	2.31	2 410	43	70	393	6 000	2.32	3 020	50
93	35	4 770	2.02	2 520	53						
97	56	5 760	2.02	3 720	64						
Averages..	42	5 370	2.11	2 883	53						
GROUP 7.											
87	35	7 965	4.10	3 080	39						

disappears, the steel does not suffice to carry the load by its lateral hold on the granular, non-coherent mass, but that with the failure of cohesion of the concrete occurs the failure of the column; and, with a larger quantity of steel than this, the failure of cohesion in the

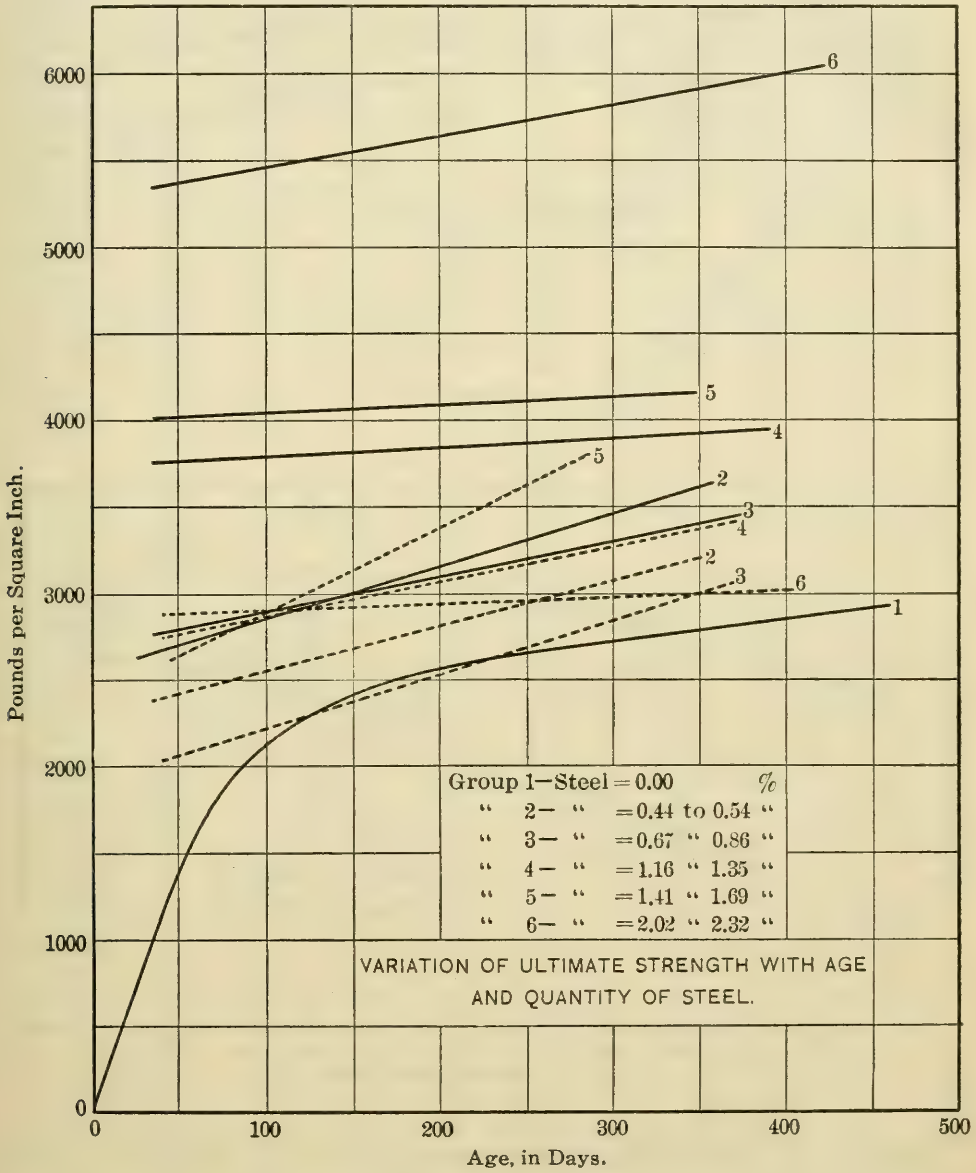

FIG. 10.

concrete does not produce failure of the column, as the steel acting on the core suffices to support the load.

Attention is here directed to the fact that this limit of 0.0158 is based on the volume ratio of steel to concrete, on wire having a yield

point and ultimate strength as shown in the table, and on a broken-stone concrete. With a weaker steel, or with aggregate material having a different coefficient of friction, this limit would be changed.

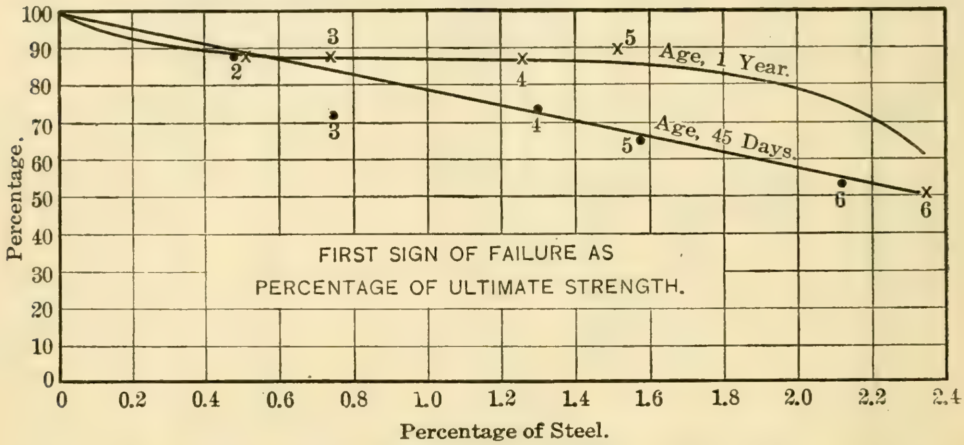

FIG. 11.

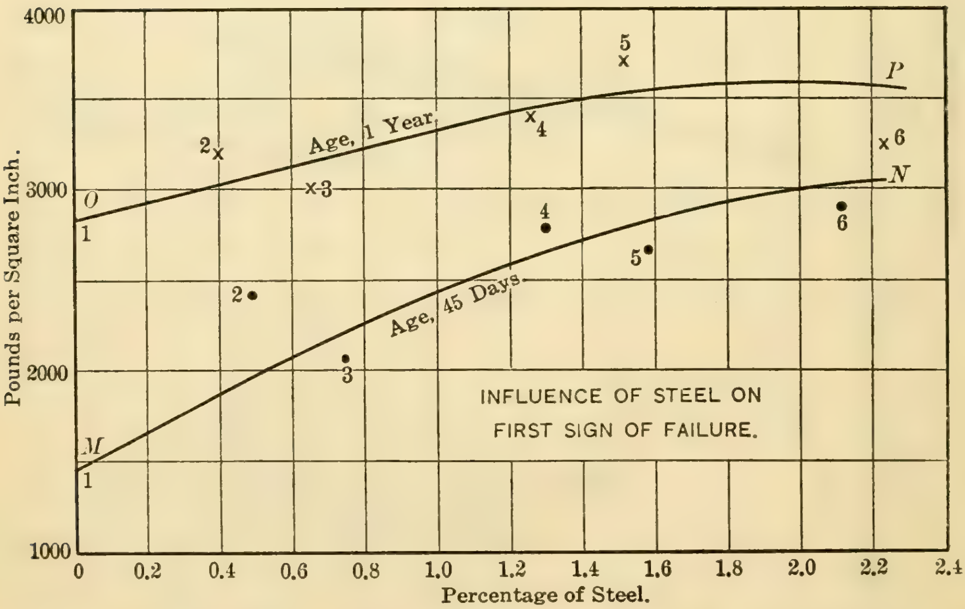

FIG. 12.

From C the curve, C-G, swings away quite slowly from C-D, drawn parallel to A-G, showing a more rapid breaking down of the cement bond as the load increases; though E-G draws away from the similar parallel, E-F, much more rapidly. Both, however, indicate clearly that the reinforcement is effective in raising the ultimate strength, even with small proportions of steel.

The ordinate to the curve at any point consists of two portions: that below *A-B*, due to the wire confining the granular mass, and that above *A-B*, due to the cohesion of the concrete; or, perhaps better, that below *A-B*, due to the reinforcing steel, and that above due to the cement bond.

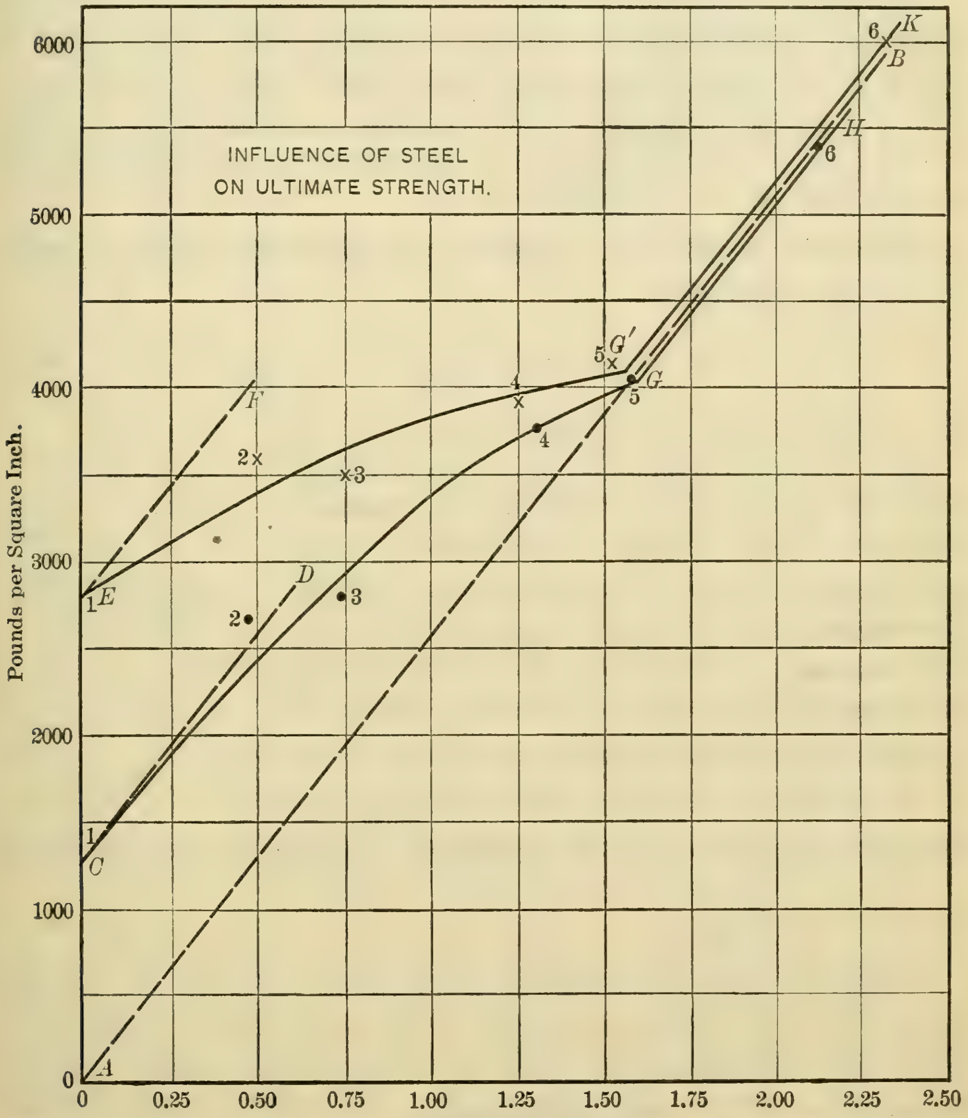

FIG. 13.

By rational methods the following is found, assuming that the column fails by shear on an oblique plane: Given a particle acted on by the forces, P and Q, at right angles to each other, producing stresses p and q on their normal planes (p being greater than q),

the resultant normal and tangential stresses on a plane, M-N, making an angle, ϕ, with the direction of Q will be

$$f_n = p \cos.^2 \phi + q \sin.^2 \phi$$
$$f_t = (p - q) \cos. \phi \sin. \phi$$

If f_z *is* the shearing resistance of the material and μ its coefficient of friction, then at any time the net resistance to slip on any plane is $\mu f_n + f_z - f_t$, and, at the instant when failure occurs, $f_t = \mu f_n + f_z$.

For a granular material, such as concrete becomes when the bond of the cement is destroyed, f_z is zero, and for such material the resistance to slip $= \mu f_n - f_t$.

The plane on which this resistance is a minimum is given in terms of μ by the expression:

$$\sin.^2 \phi = \frac{1}{2} \pm \frac{1}{2}\sqrt{1 - \frac{1}{1 + \mu^2}},$$

and is independent of the relative values of p and q.

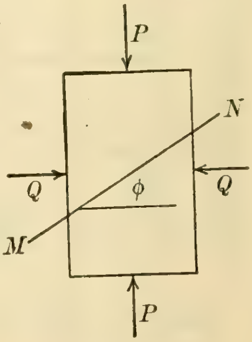

FIG. 14.

In a hooped column, the steel fails in tension when the column fails. Whether the ultimate strength of the column is determined by the yield point of the steel or its ultimate strength, is debatable, but the former appears to be the more rational. Let f_s denote the tensile stress in the hooping at the instant of failure. When the volume of the hooping steel is n times the volume of the concrete, the unit radial compression in the concrete produced by the hooping is $\frac{n}{2}$ times the unit tension in the steel, or $q = \frac{n}{2} f_s$.

Assuming, from consideration of the curves of Fig. 13, that the cohesion of the concrete disappears at the point (0.0158, 4 000), it is found that a coefficient of friction, $\mu = 0.95$, corresponding to a friction angle of 43° 30′ gives a compression, $q = 740$ lb. and $f_s = 93\,800$ lb., approximating the ultimate strength of the hooping, but if $\mu = 1.00$ and the friction angle is 45°, $q = 680$ lb. and $f_s = 86\,000$ lb., or about the yield point of the spiral. It would seem, therefore, that, with a steel having a relatively high yield point, the question whether ultimate strength or yield point determines the strength of the column

is not an important one, because so small a variation in the angle of friction corresponds to so large a range of stress in the steel.

This method of treatment takes no account of flexure in the column, and, therefore, is applicable only to short columns.

The writers do not consider that the tests prove that the curves of Fig. 13 are typically correct. A large number of tests, giving more complete groups, especially in the higher percentages of reinforcement, would be necessary for a final conclusion; but the tests as given indicate a break in the continuity of the curve, as at *G*, and, as the rational analysis leads to a not unreasonable value of the friction coefficient in the aggregate, the curves are presented for consideration.

The curve, *C-G-H*, of Fig. 13 is reproduced in Fig. 15 for the purpose of comparing with it the results of the tests on Columns 15 to 22, inclusive, built by setting the plain column on centers in a lathe and winding the reinforcement on under constant tension, as described previously. The results of the tests of these individual columns are shown, with a fair curve drawn through them. As these were tested at 28 days, the initial point of the curve is taken at 900 lb., as determined from Curve 1 on Fig. 10. From these few tests it would appear that initial tension in the reinforcement gives higher ultimate strength and also a relatively greater effect from the cohesion of the concrete, as shown by the greater angle of rise of the curve in its lower portion. The soundness of the latter conclusion is doubted, however, it seeming to be more probable that this difference is due to curing these columns in air instead of water, thus developing a higher strength at this age. It is indicated plainly, however, that the initial stress in the hooping prevented the weakening of the concrete bond until the ultimate strength of the steel was nearly reached, when steel and concrete failed together. If the fabrication of columns of this type were practicable in actual construction it would evidently offer some advantage in strength.

The usefulness of this group of tests consists in the demonstration, so far as it may be considered conclusive, that the destruction of the bond of the concrete itself is restrained by the hooping, so that this destruction becomes a gradual process, instead of the abrupt one which occurs in the plain column, and that this restraining effect is produced, even with small percentages of steel.

On Fig. 15 are plotted also the points for Columns 10 to 14, inclusive, marked *z*; also from tests* by A. N. Talbot, M. Am. Soc. C. E., Columns 171, 172, 173, 181, 182, 183, reinforced with high-carbon steel, and marked *T*, and Columns 176, 177, 178, 186, 187, reinforced

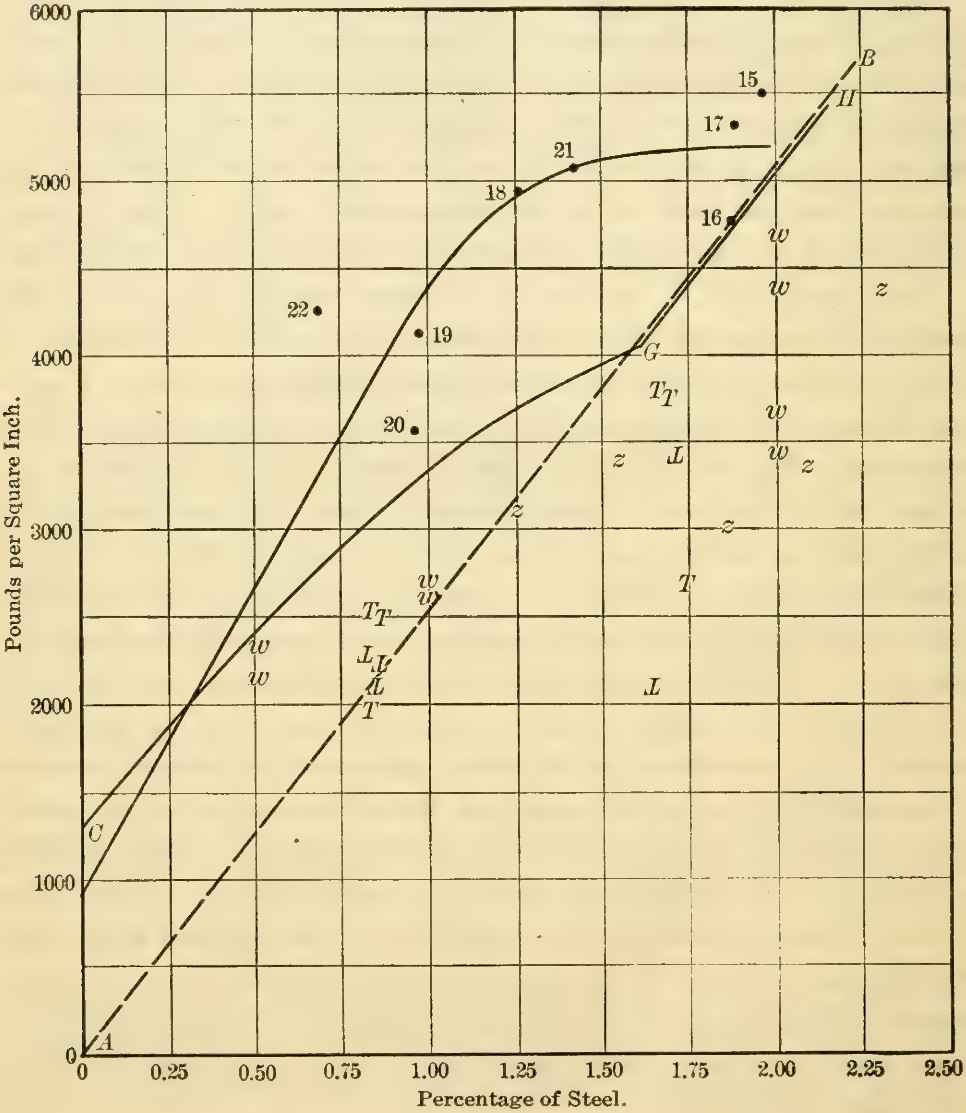

FIG. 15.

with mild steel and marked *L*; also from tests† by Professor M. O. Withey, Columns *C*1, *C*2, *C*3, *C*4, *H*1, *H*2, *L*1, *L*2, marked *w*. All the columns of this group are reinforced with hooping prepared by commercial firms, and differ only in minor details. Compared with

* Bulletin No. 20, Illinois Experiment Station.

† Bulletin No. 466, University of Wisconsin.

Columns 15 to 115 of this series, they differ first in having longitudinal or spacing members relatively much heavier, and second in the use of a heavier hooping wire spaced on a higher pitch.

The fact, noted by Considère, that a low-pitch spiral is more effective than one of high pitch is brought out clearly here. These points lie in a fairly close group, but all are well below the curve, *C-G-H*.

There is no reason to believe that these lower values are due to inferior concrete, nor, on account of their consistent grouping, can it be argued that they are due to differences in conditions of the tests, unless, because of the fact that these were cured in air and the *C-G-H* columns were cured in water. A consideration of Fig. 10, however, shows the line of Group 2, with 0.44 to 0.54% of steel, lying above that of Group 3, with 0.67 to 0.86 per cent. It happens that all the columns of Group 2 were reinforced with the 0.061 wire, and that Groups 3, 4, and 5, have each only one column with this fine wire, all other reinforcement being heavier. The writers, therefore, believe that the points under discussion lie below *C-G-H,* because of the fact that the spiral is of coarser wire and consequently higher pitch.

Values of s_2.—The stress-strain curves show a permanent deformation which is greater in the short-time tests. This is shown by arranging in two groups, the first including all columns from 84 to 115, all of which were loaded under the extensometer to 700 lb. per sq. in., and all at approximately the same age. These show on the 100-lb. line a length of 53 units (of $\frac{1}{600\,000}$ in.) between the ascending and descending curves. The second group includes Columns 46, 48, 51, 52, 53, 54, 60, 62, 64, 67, 69, 71, 73, 74, 75, 79, 80, 81, and 82, all but four of which were loaded to 1 400 lb., and all above 1 150 lb. have an average age of 352 days and show on the 100-lb. line a length of 35 units.

In the majority of cases these curves show a satisfactory uniformity, but some exceptions appear, as Column 72, tested at 388 days, which shows an abnormal lack of resilience, the load dropping from 1 400 to 1 000 lb. before recovery began.

For examining these results to determine the influence of the hooping on them, the columns are grouped in Table 5 in the same manner as for ultimate strength. The strains here are read off from the plotted curves, instead of being taken from the tables.

The average values of these strains, using ages in days as ordinates and strains as abscissas, are plotted on the diagram, Fig. 16, where

the numerals show the group numbers. Separate curves are shown for stresses of 400 to 900 lb. per sq. in. Here appears the unexpected condition that the plain columns show less strain than those which are reinforced. Before beginning the tests the writers had expected that, at least in the higher stresses, the columns would show, both in the s_2 and the m, just the opposite. If the strain in the reinforced columns had been less than in the plain ones, or if no difference between the two had appeared, the explanation would be simple; but, that the presence of a spiral of steel should be accompanied by a greater deformation

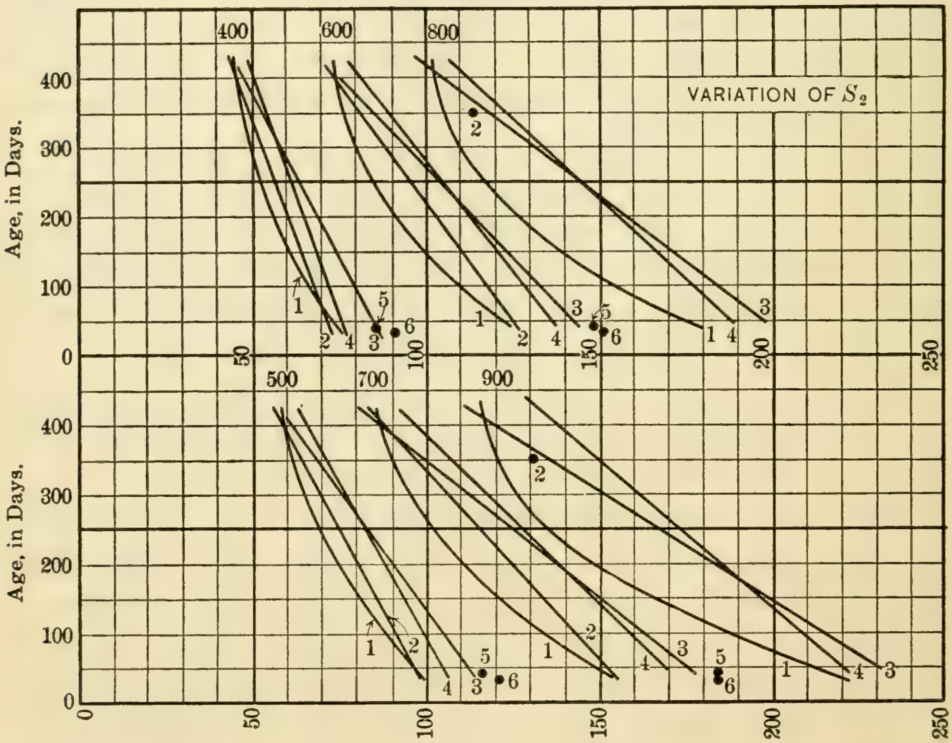

FIG. 16.

does not find a ready reason, unless it be that accidental peculiarities have existed which have escaped observation, but which, nevertheless, were sufficient to produce the condition shown. The consistency with which this runs through all groups leads to the opinion that the same results would be found by a repetition of the tests. That the shrinkage of the concrete, causing initial compression in the steel, might be the cause is untenable, first, because the concrete was immersed in water during the entire aging period, and the shrinkage, therefore, would not occur so as to produce this result; and second, because if the steel spiral were under initial compression one would expect to find

TABLE 5.—Values of s_2.

Group 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.......	42	79	108	135	163	......	......	......	......	......	......	......
114.......	35	68	91	115	144	......	......	......	......	......	......	......
115.......	35	65	90	115	142	......	......	......	......	......	......	......
Average....	37	.71	96	122	150	......	......	......	......	......	......	......
50.......	65	70	91	113	136	161	......	......	......	......	......	......
38.......	63	81	104	128	158	188	......	......	......	......	......	......
42.......	58	68	90	113	135	160	......	......	......	......	......	......
43.......	73	85	112	143	176	213	251	......	......	......	......	......
44.......	74	78	103	131	160	192	227	......	......	......	......	......
58.......	61	67	90	114	140	168	197	......	......	......	......	......
61.......	77	68	88	108	130	153	177	......	......	......	......	......
Average....	67	74	97	121	148	176	213	......	......	......	......	......
46.......	296	47	65	83	98	114	132	149	167	......	......	......
47.......	288	72	89	106	122	138	153	......	......	......	......	......
53.......	295	42	54	66	79	92	105	119	132	......	......	......
67.......	322	40	53	67	82	97	112	128	143	160	177	196
Average....	300	50	65	80	95	110	125	132	147	160	177	196
48.......	414	36	48	59	71	84	97	110	123	136	148	161
51.......	434	38	51	63	75	88	100	114	128	142	156	170
52.......	440	44	57	71	85	99	113	128	142	156	170	184
54.......	441	43	54	66	78	90	101	114	128	142	156	170
62.......	434	53	68	82	96	112	128	145	161	178	194	212
79.......	416	45	60	75	91	107	123	140	158	178	198	......
80.......	417	59	75	94	112	130	150	170	192	215	240	264
Average....	428	45	59	73	86	101	116	132	146	164	180	193

Group 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
111....	35	0.50	73	99	127	155	......	......	......	......	......	......	......
71....	317	0.45	51	64	78	92	106	121	137	154	172	190	211
74....	370	0.40	62	80	99	119	138	157	175	192	210	227	......
75....	300	0.54	38	54	70	86	103	120	139	159	181	205	230
82....	413	0.58	48	61	76	91	108	125	143	162	182	202	223
Average.	350	0.49	50	65	81	97	114	131	148	167	186	206	221

TABLE 5.—VALUES OF s_2.—(*Continued*).

GROUP 3.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
66....	74	0.72	83	107	133	161	191	225	.....	......	......	......	......
101...	29	0.71	89	120	153	190	......	......	......	......	......	......	......
107....	35	0.78	83	113	147	179	......	......	......	......	......	......	......
Average.	46	0.74	85	113	144	177	191	225	......	......	......	......	......
60....	311	0.72	66	81	96	111	127	144	162	181	200	220	241
64....	435	0.67	44	59	74	89	105	122	140	158	177	198	218
69....	323	0.72	53	70	86	104	123	143	164	186	208	233	260
72....	388	0.72	57	73	89	105	123	141	160	181	205	230	259
81....	270	0.86	48	66	83	100	120	140	160	181	......	......	......
Average.	345	0.74	54	70	86	102	120	138	157	177	197	220	244

GROUP 4.

59....	67	1.31	76	101	128	157	186	219	256	......	......	......	......
99....	31	1.16	76	110	146	183	......	......	......	......	......	......	......
Average.	49	1.23	76	105	137	170	186	219	256	......	......	......	......
63....	395	1.24	55	70	86	101	117	133	150	169	188	......	......
73....	428	1.26	46	60	74	89	104	120	135	152	169	188	206
Average.	411	1.25	50	65	80	95	110	126	142	160	178	188	206

GROUP 5.

56....	58	1.69	84	115	148	184	227	......	......	......	......	......	......
92....	34	1.41	90	121	156	193	.....	......	......	......	......	......	......
93....	34	1.41	89	120	153	187	......	......	......	......	......	......	......
94....	35	1.41	83	112	142	175	......	......	......	......	......	......	......
105....	35	1.35	91	125	161	202	......	......	......	......	......	......	......
106....	35	1.35	77	104	133	164	......	......	......	......	......	......	......
Average.	38	1.44	86	116	149	184	227	......	.....	......	......	......	......

GROUP 6.

89....	35	2.30	90	118	144	172	......	......	......	.....	......	......	......
95....	35	2.04	94	124	160	199	......	......	......	......	......	......	......
96....	35	2.02	88	120	153	180	......	......	......	......	......	......	......
Average.	35	2.12	91	121	152	184	......	......	......	......	......	......	......

m, at short time, less for reinforced than for plain columns, whereas the curves for m show precisely the opposite. The only explanation which the writers can suggest is that the presence of the reinforcement, with its rather closely spaced wires, may have prevented in a measure the thorough elimination of bubbles and small voids, thus rendering the concrete as a whole less dense. Moreover, Professor Talbot, has discussed* the same condition, which he found to exist in his experiments, and on which he places the same interpretation.

These curves indicate that, within the limits of ordinary working stresses, spiral reinforcement does not noticeably lend assistance to the concrete in carrying the load, but that, if such assistance is developed, it is at stresses well above the ordinary working limits.

Values of e.—The values of e have been computed and plotted for the ascending curves only. In Table 6 the columns are grouped as before, and the values are as read from the plotted curves.

The average values from Table 6 are plotted on Fig. 17, the ages, in days, being the ordinates and the values of e the abscissas. The curves show a close agreement for both plain and reinforced columns at 425 days, and separate more widely with decreasing age and increasing stress. As these are derived directly from s_2, they show the same general relations between the plain and reinforced columns, and the comments made regarding the stress-strain curves apply with equal force here. The evidence of these curves is decidedly against the use of values of e greater than 10 for computing the strength of reinforced columns. It runs above that value only for the combinations of short time with high stresses, a combination to be avoided in practice as much as possible.

Values of m.—m is the ratio of longitudinal to lateral deformation, the two being opposite in kind when the member is stressed in one direction only, and its value, m, in terms of the observed quantities, becomes

$$m = \frac{\lambda_2}{\lambda_1} = \frac{5\ D\ s_2}{3\ s_1}.$$

This involves one more observed quantity, s_1, which is more difficult to measure, not only because it is very small, but also because of the likelihood of the instrument slipping. The curves show less

* Bulletin No. 20, Illinois Engineering Experiment Station.

TABLE 6.—Values of *e*.

Group 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.......	42	9.4	10.2	10.8	11.2	......	......	......	......	......	......	......
114.......	35	8.3	8.9	9.4	9.8	......	......	......	......	......	......	......
115.......	35	7.8	8.6	9.3	9.9	......	......	......	......	......	......	......
Average....	37	8.5	9.2	9.8	10.3	......	......	......	.....	......	......	......
38.......	63	9.8	10.1	10.4	10.8	11.5	......	......	......	......	......	......
42.......	58	8.3	8.8	9.2	9.4	9.6	......	......	......	......	......	......
43.......	73	10.4	10.9	11.4	12.0	12.7	13.3	.....	......	......	......	......
44.......	74	9.6	10.1	10.5	11.0	11.5	12.2	......	......	......	......	......
50.......	65	8.4	8.8	9.1	9.4	9.8	......	......	......	......	......	......
58.......	61	8.0	8.6	9.2	9.7	10.2	10.7	......	......	......	......	......
61.......	77	8.2	8.5	8.8	9.0	9.3	9.6	.....	......	......	......	......
Average....	67	8.9	9.4	9.8	10.2	10.7	11.4	......	......	......	......	......
46.......	296	5.7	6.2	6.6	6.9	7.0	7.1	7.2	7.4	......	......	......
47.......	288	8.7	8.6	8.5	8.4	8.3	8.2	......	......	......	......	......
53.......	295	5.1	5.2	5.3	5.5	5.6	5.7	5.8	5.9	......	......	......
67.......	323	4.9	5.1	5.3	5.5	5.7	5.9	6.1	6.3	6.4	6.5	6.6
Average....	300	6.1	6.3	6.4	6.6	6.7	6.7	6.4	6.5	6.4	6.5	6.6
48.......	414	4.3	4.5	4.8	5.0	5.1	5.2	5.4	5.5	5.5	5.5	5.5
51.......	434	4.9	5.0	5.1	5.2	5.3	5.4	5.5	5.7	5.8	5.9	6.0
52.......	440	5.4	5.7	5.8	5.9	6.0	6.0	6.1	6.1	6.2	6.2	6.3
54.......	441	5.0	5.1	5.2	5.3	5.4	5.5	5.5	5.6	5.7	5.8	5.9
62.......	434	6.4	6.5	6.6	6.7	6.8	6.9	7.0	7.1	7.1	7.2	7.3
79.......	416	5.4	5.7	6.0	6.2	6.4	6.6	6.8	7.0	7.2	7.4	7.5
80.......	417	7.0	7.2	7.4	7.6	7.8	8.0	8.2	8.4	8.6	8.8	9.0
Average....	428	5.5	5.7	5.8	6.0	6.1	6.2	6.4	6.5	6.6	6.7	6.8

Group 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100
111.........	35	0.47	8.9	9.6	10.2	10.8	......	......	......	......
71.........	317	0.45	6.1	6.2	6.3	6.4	6.5	6.5	6.6	6.7
74.........	370	0.44	7.7	7.9	8.1	8.2	8.3	8.3	8.3	8.3
75.........	300	0.54	4.6	5.1	5.5	5.9	6.2	6.4	6.7	7.0
82.........	413	0.52	5.8	6.0	6.2	6.4	6.6	6.8	7.0	7.1
Average......	350	0.47	6.1	6.3	6.5	6.7	6.9	7.0	7.1	7.3

TABLE 6.—(*Continued*).

GROUP 3.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100
66.........	74	0.72	10.1	10.4	10.7	11.0	11.4	12.0	......	......
101.........	29	0.71	10.7	11.5	12.3	13.0	......	......	......	......
107.........	35	0.78	10.1	11.0	11.8	12.2	......	......	......	......
Average......	46	0.74	10.3	11.0	11.6	12.1	11.4	12.0	......	......
60.........	311	0.72	8.1	7.9	7.8	7.8	7.8	7.8	7.9	8.0
64.........	435	0.67	5.2	5.6	5.9	6.1	6.3	6.5	6.7	7.0
69.........	323	0.72	6.5	6.9	7.1	7.3	7.4	7.7	8.0	8.2
72.........	388	0.80	6.8	7.0	7.1	7.3	7.4	7.6	7.8	8.0
81.........	270	0.86	5.7	6.1	6.6	6.9	7.2	7.4	7.6	7.8
Average......	344	0.74	6.5	6.7	6.9	7.1	7.2	7.4	7.6	7.8

GROUP 4.

59.........	67	1.31	9.2	9.8	10.4	10.9	11.3	11.8	12.2	......
99.........	31	1.16	9.1	10.5	11.8	12.8	......	......	......	......
Average......	49	1.23	9.1	10.1	11.1	11.8	11.3	11.8	12.2	......
63.........	395	1.24	6.6	6.7	6.8	6.9	7.0	7.1	7.3	7.4
73.........	428	1.26	5.5	5.8	6.0	6.2	6.3	6.5	6.6	6.7
Average......	411	1.25	6.0	6.1	6.4	6.5	6.6	6.8	6.9	7.0

GROUP 5.

56.........	58	1.69	10.1	10.9	11.8	12.7	13.6	14.7	15.9	......
92.........	34	1.41	10.6	11.7	12.5	13.3	......	......	......	......
93.........	34	1.41	10.8	11.5	12.2	12.9	......	......	......	......
94.........	35	1.41	10.0	10.8	11.4	12.1	......	......	......	......
105.........	35	1.35	10.9	12.0	13.0	14.0	......	......	......	......
106.........	35	1.35	9.5	10.2	10.8	11.3	......	......	......	......
Average......	38	1.44	10.3	11.2	11.9	12.7	13.6	14.7	15.9	......

GROUP 6.

89.........	35	2.30	10.9	11.4	11.7	11.9	......	......	......	......
95.........	35	2.04	11.3	12.0	12.8	13.8	......	......	......	......
96.........	35	2.02	10.5	11.5	12.4	13.1	......	......	......	......
Average......	35	2.12	10.9	11.6	12.3	12.9	......	......	......	......

uniformity than those of s_2 and e, and a few have been thrown out of the general averages where they appeared to differ widely and erratically from the others of the group. In Table 7 the columns are grouped as before.

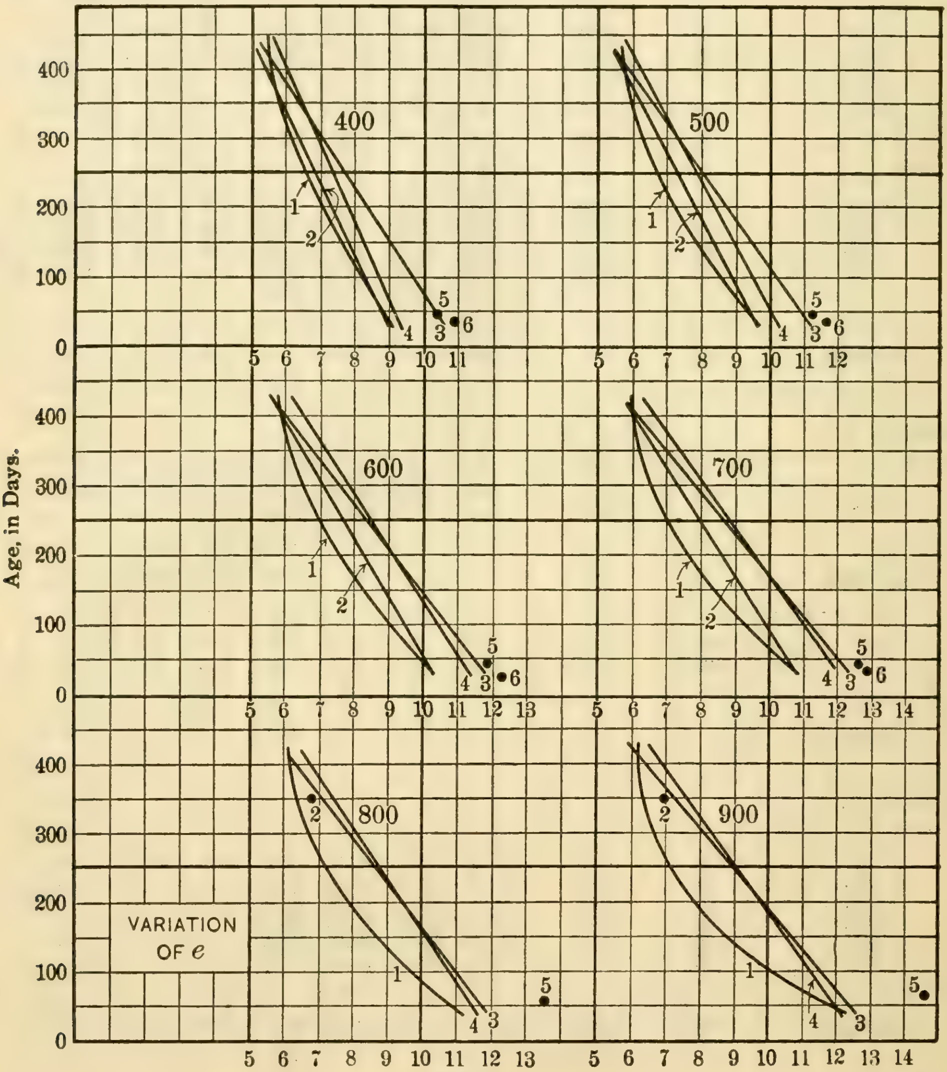

FIG. 17.

The average values from Table 7 are plotted on Fig. 18; with ages, in days, as ordinates, and values of m as abscissas. The single column of Group 2, tested at short time, gives values which for the lower stresses appear to be too high, but the other points lie fairly close together, the value 7 for long time and 8 for short time representing fair averages. On the 800 and 900-lb. curves the value for long time drops below 7. In the few cases where this constant has been used in

discussions, as noted by the writers, it has been assumed as equal to 4, the value derived by Grashof under certain assumptions made by Saint Venant. Estimates based on this would evidently be considerably in error. The value, 7-8, as here found is not surprising when compared with the value for steel, which is about 4.

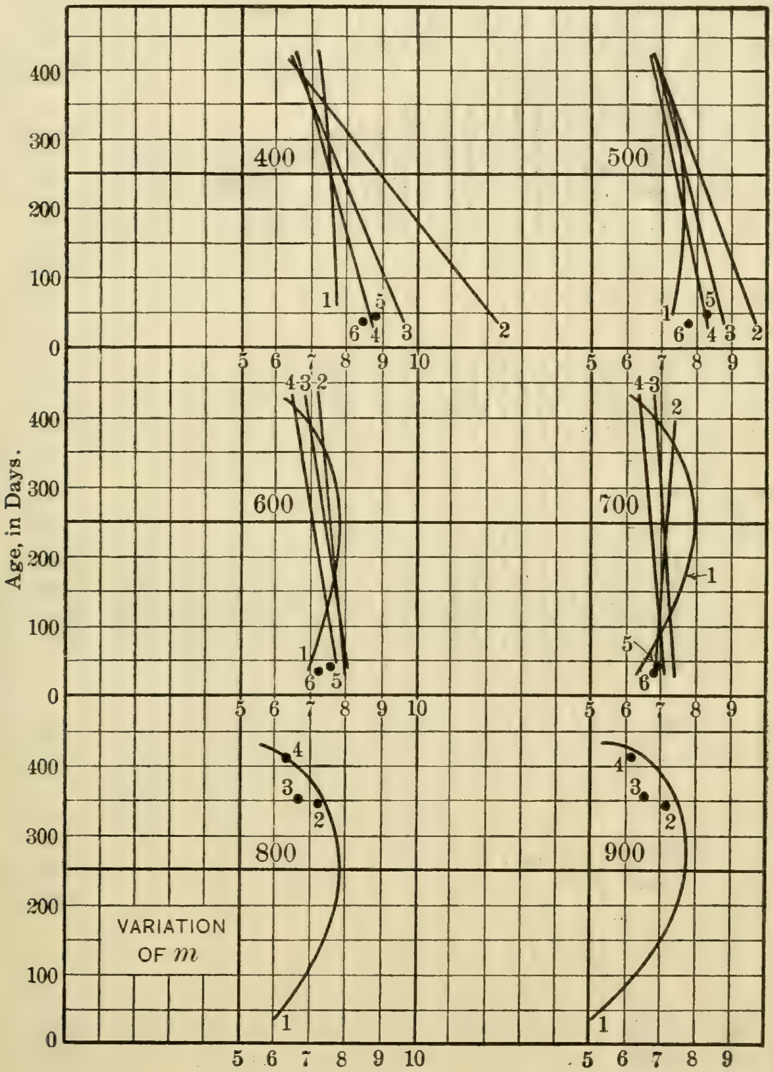

FIG. 18.

As compared with steel, concrete is lacking in density, has numerous small voids, and is granular in nature, with possibly a state of tension existing in the interstitial masses of cement due to the shrinkage of the latter in setting, under certain conditions. The first effect of applying a compressive stress to it would probably be to decrease

TABLE 7.—VALUES OF m.

GROUP 1.

Column.	Age.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
84.......	42	6.5	5.0	5.0	4.9	......	......	......	......	......	......	......
114.......	35	7.6	7.5	7.0	6.6	......	......	......	......	......	......	......
115.......	35	3.1	3.3	3.5	3.5	......	......	......	......	......	......	......
Average....	37	5.8	5.2	5.1	5.0	......	......	......	......	......	......	......
38.......	63	8.7	8.4	8.2	7.7	7.1	......	......	......	......	......	......
42.......	58	6.1	6.6	6.7	6.9	6.9	......	......	......	......	......	......
43.......	73	5.5	5.1	4.6	4.3	4.1	3.9	......	......	......	......	......
44.......	74	10.1	9.3	8.7	8.1	7.3	6.6	......	......	......	......	......
50.......	65	7.7	7.4	7.2	7.0	6.8	......	......	......	......	......	......
58.......	61	10.3	9.1	8.2	7.5	6.9	6.4	......	......	......	......	......
61.......	77	9.2	8.6	8.1	7.9	6.9	6.3	......	......	......	......	......
Average....	67	8.2	7.8	7.4	7.0	6.6	5.8	......	......	......	......	......
46.......	296	5.7	6.7	7.5	8.1	8.4	8.4	8.7	8.7	......	......	......
47.......	288	4.2	4.6	4.8	5.0	5.1	5.0	......	......	......	......	......
53.......	295	10.5	10.7	10.5	10.3	10.0	9.5	9.0	8.5	......	......	......
67.......	323	9.0	8.0	7.9	7.9	7.9	7.9	7.9	7.9	7.8	7.5	7.2
Average....	300	7.4	7.5	7.7	7.8	7.8	7.7	8.4	8.3	7.8	7.5	7.2
48.......	414	5.5	3.7	3.3	3.0	2.3	2.1	1.9	1.7	1.6	1.5	1.5
51.......	434	7.2	6.9	6.6	6.3	6.2	6.1	6.0	6.0	5.9	5.8	5.7
54.......	441	7.6	7.3	7.0	6.7	6.1	5.9	5.6	5.5	5.4	5.3	5.2
62.......	434	10.9	10.2	9.9	9.5	9.0	8.9	8.7	8.4	8.2	8.0	7.8
79.......	416	5.2	5.2	5.1	5.1	5.0	4.9	4.8	4.7	4.7	4.6	4.5
80.......	417	6.4	6.7	6.7	6.6	6.4	6.2	6.0	5.8	5.5	5.3	5.0
Average....	426	7.1	6.7	6.4	6.2	5.8	5.7	5.5	5.4	5.2	5.1	5.0

GROUP 2.

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
111....	35	0.47	12.5	9.5	7.8	6.9	......	......	......	......	......	......	......
71....	317	0.45	8.7	8.7	8.6	8.4	8.1	8.0	8.0	8.0	8.0	7.9	7.8
75....	300	0.54	8.3	8.2	8.1	8.0	7.9	7.8	7.7	7.5	7.3	7.0	6.8
82....	413	0.52	5.2	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.3	5.3	5.3
Average.	343	0.50	7.4	7.4	7.4	7.3	7.2	7.1	7.0	7.0	6.9	6 7	6.6

the volume by linear deformation in the direction of the stress, with little or no lateral enlargement. If this should actually take place, one would expect the value of m to be infinite at the beginning of application of load, and to decrease as the load increased, giving a

TABLE 7.—(*Continued.*)

Column.	Age.	Percentage of steel.	400	500	600	700	800	900	1 000	1 100	1 200	1 300	1 400
Group 3.													
66....	74	0.72	14.4	13.3	12.6	12.0	11.6	11.2	......	......	......	......	......
101....	29	0.71	8.7	7.8	7.1	6.2	......	......	......	......	......	......	......
107....	35	0.78	5.7	5.1	4.7	4.2	......	......	......	......	......	......	......
Average.	46	0.74	9.6	8.8	8.1	7.5	11.6	11.2	......	......	......	......	......
64....	435	0.67	5.7	5.7	5.8	6.0	5.9	5.8	5.8	5.8	5.7	5.6	5.5
69....	323	0.72	7.9	8.0	7.8	7.2	6.6	6.3	6.1	6.0	5.8	5.7	5.5
72....	383	0.72	7.8	7.8	7.6	7.4	7.2	7.0	6.8	6.6	6.4	6.1	......
81....	270	0.86	6.6	7.0	7.7	7.2	7.1	7.0	6.8	6.5	......	......	......
Average.	352	0.74	7.0	7.1	7.2	7.0	6.6	6.5	6.4	6.2	6.0	5.9	5.5
Group 4.													
59....	67	1.31	11.6	10.8	9.9	8.9	8.0	7.3	6.7	......	......	......	......
99....	31	1.16	6.0	5.9	5.7	5.3	......	......	......	......	......	......	......
Average	49	1.23	8.8	8.3	7.8	7.1	8.0	7.3	6.7	......	......	......	......
63....	395	1.24	5.3	5.3	5.2	5.2	5.1	5.1	5.0	5.0	5.0	......	......
73....	428	1.26	8.1	8.1	8.0	7.9	7.7	7.4	7.1	7.0	7.0	6.9	6.9
Average.	411	1.25	6.7	6.7	6.6	6.5	6.4	6.2	6.0	6.0	6.0	6.9	6.9
Group 5.													
56....	58	1.69	9.1	9.0	8.8	8.5	8.1	7.6	7.0	6.3	......	......	......
92....	34	1.41	10.1	9.0	8.0	7.2	......	......	......	......	......	......	......
93....	34	1.41	10.4	9.3	8.3	7.3	......	......	......	......	......	......	......
94....	35	1.41	8.2	7.7	7.2	6.6	......	......	......	......	......	......	......
105....	49	1.35	7.9	7.7	7.1	6.6	......	......	......	......	......	......	......
106....	44	1.50	7.5	7.0	6.4	5.9	......	......	......	......	......	......	......
Average.	42	1.46	8.9	8.3	7.6	7.0	8.1	7.6	7.0	6.3	......	......	......
Group 6.													
95....	35	2.04	10.4	9.6	9.0	8.4	......	......	......	......	......	......	......
96....	35	2.02	9.9	8.5	8.0	8.0	......	......	......	......	......	......	......
Average.	35	2.03	10.2	9.1	8.5	8.2	......	......	......	......	......	......	......

curve such as is found in Columns 52 or 62. Generally, however, the curve runs from a lower value up to a maximum at a stress which roughly averages 275 lb. per sq. in., but which may be anywhere between 100 and 500 lb., or may not be clearly defined at all. The

points up to the 500-lb. stress are irregular in location, and it is often impracticable to lay a fair curve through them, indicating that the material is settling together somewhat irregularly and adjusting itself under the imposed load, not uniformly, as would a homogeneous mass, but somewhat spasmodically, until a stage is reached where the mass is thoroughly compacted, and all the initial stresses are relieved; from this point the deformation becomes quite consistent, as is shown by the values of m becoming quite regular, the points varying but slightly from a fair curve.

This same tendency to irregularity is shown slightly in the s_2 curve, but far more plainly in the e curve, where the points are irregularly placed, frequently showing a considerable break in continuity at some point. This irregularity of e seems to be without any law, except that it disappears at stresses of from 200 to 400 lb. There is no reason to suppose that slipping of the extensometer would be more likely to occur here than at higher stresses, and the fact that the values of e, which depend only on the s_2 readings, show an irregularity similar to that of m would indicate that this is not due to slip in the instrument.

It is the conclusion of the writers, therefore, that below about 400 lb. per sq. in., the concrete deforms under the load quite irregularly and without any law; that at about 400 lb. the causes of this irregularity are largely eliminated, and that thereafter the action conforms closely to a definite law which can be represented by a fair curve until some higher limit is reached; which limit lies well beyond ordinary working stresses, and is not defined by these tests. At this limit the destruction of the cement bond would begin, and the curves would become irregular again.

Conclusions.

From the foregoing results the writers deduce the following conclusions, as applicable to the materials used in these tests:

(1) The value of Poisson's ratio is approximately 8 for short-time and 7 for long-time tests.

(2) The value of e is in general below 10, only exceeding that value for short time and high stress.

(3) Spiral reinforcement does not, by its restraining effect on the core, perceptibly affect the values of m or e for stresses below 900 lb. per sq. in.

(4) Spiral reinforcement does not perceptibly assist the concrete in carrying the load within the limits of ordinary working stresses.

(5) The limit at which the first visible sign of failure occurs is raised by spiral reinforcement. This effect is greatest in short-time tests, and is perceptible for all percentages of reinforcement.

(6) The plain column fails abruptly, giving no warning of approaching collapse. The hooped column may be depended on to give warning of approaching failure at from 70 to 90% of the ultimate strength, when not less than 0.50% of steel is used.

(7) The deformation is quite irregular up to about 400 lb. per sq. in.; above this it is quite regular up to a limit which is not defined by these tests, but is well above allowable working stresses.

The tests indicate, but cannot be said to prove conclusively, that:

(8) The ultimate strength of the column is increased by spiral reinforcement by an amount roughly proportional to the quantity of steel used, up to the limit of 1.58%; and above this limit the ultimate strength is simply the amount which can be carried by the granular core supported by the spiral, but with no cohesion of its own.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By Messrs. Onward Bates, Jared How, W. H. Courtenay, V. K. Hendricks, E. W. James, W. W. K. Sparrow, William J. Wilgus, J. E. Gibson, H. C. Vensano, William W. Crehore, and C. P. Howard.

Onward Bates, Past-President, Am. Soc. C. E. (by letter).—The valuation of public utilities for the purpose of rate-making is one of the great economic questions of the present day in the United States, affecting as it does the value of investments aggregating thousands of millions of dollars, and the income from wages and dividends of some millions of people. The able report submitted by the eminent members of this Committee will carry great weight, and will doubtless be quoted back and forth by those who are engaged in the practical solution of the problems involved. This report, however, is admittedly incomplete, and is submitted with the reservation of the privilege to add to it. Consequently, it should not be taken as the last words of the authors. Mr. Bates.

The Government of the greatest nation in the world is committed to the policy of the regulation of big business, and incidentally of little business, and we have the example of men without business experience directing in detail how business shall be conducted, holding over the heads of citizens engaged in business the threat that the Government itself will take business out of the hands of its owners and conduct it as a Government monopoly. The notable present instance of this policy is the regulation of railway rates by the Interstate Commerce Commission, and it is in the minds of many people interested in rail-

* This is a discussion on the Report of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, presented to the Annual Meeting, January 21st, 1914.

Mr. Bates. ways that a continuation of this policy will ruin this line of business endeavor, in so far as the interests of the men, women, and children who are railway stockholders are concerned.

The future of all public utilities is dependent on the rates which the public, through its Governmental instrumentalities, may establish from time to time. "Justice" will be the watchword of those who name the values and of those whose property is valued. The Committee, under the heading "General Principles Involved in Valuation for Rate-Making," writes:

"The primary requirement in the valuation of a public service property is that justice shall be done both to the owner of the property and to the public." * * * "The exercise of the power to regulate rates should be based on equity to both parties.

"It has been well settled by the highest Courts that the owner of such property is entitled to a fair return upon a fair value of the property utilized in or reasonably necessary to the service."

Justice, however, is often obscured. It appears from the foregoing quotation that the highest Courts depend for justice on the fairness of the valuation. It is to be expected, and it is certain to happen, that controversies will arise between the owners of property and those who place values on such properties, and it is only natural and altogether probable that partisanship and bitterness will be engendered between the representatives of property interests and of the authority which undertakes to establish the value of property.

At this point we meet what the writer considers one of the worst features of Governmental ownership—two parties disputing over values, of which, one party, with the prestige of Governmental authority behind him, is clothed with the power to do injustice, while the other party is practically helpless. Justice, as has been stated, depends on the fairness of the party to the controversy which holds the power of decision, and equity can only be secured by having the valuer fair and just and competent.

We are all prone to evil. Men are not perfect, and when engaged in a fight are inclined, during the heat of action, to forget the principles of equity and justice while endeavoring to secure an advantage over an opponent. There is an additional danger to vested interests to be expected from the appointment of incompetent valuers. This is the day of professional reformers and efficiency enthusiasts, most of whom are not under the conservative and sobering influence of personal investments which are at stake. At a time when the Government is planning an organization for the purpose of valuing public utilities, which will require a great number of men, most of whom should be experts, and with the condition that such qualified experts are generally in the employment of the public utility corporations, it is to be expected that men with bright minds and plausible language will find in this organi-

zation a market for their talents. These talents may be very great, but, if not balanced by experience and a sense of responsibility toward the interests at stake, may be dangerous instruments in the hands of the Government, leading to great injustice toward its citizens. Mr. Bates.

An illustration of what may happen in this line is to be found in the seventy-eight questions submitted by the Interstate Commerce Commission to the railways as a preliminary to the granting of an increase in rates. These questions are too long to be quoted here, but the necessity for an increase in rates seems to have been lost sight of by the Commission, which has started an investigation entirely aside from the business question to be determined, with the only possible result that it will create a delay in the administration of justice.

If it be assumed that, in the future, railway rates will be, or may be, based on the valuation of the physical property of railway corporations, we will find a large proportion of the membership of this Society engaged in this work of valuation, and divided between a staff of engineers employed by the Interstate Commerce Commission for this purpose, on the one side, and the engineers employed by the railway corporations, on the other side. Engineers are already being lined up in these two classes, between which there will be great diversity of opinion on many points, notably on the questions of depreciation and land values. Under these circumstances it will be most unfortunate for the report of the Committee to be considered as anything more than an expression of the opinions of its distinguished authors. The writer is willing to be corrected if he is wrong in making the statement that the Committee was not appointed to make a report on valuation for the purpose of rate-making. Its function was "to formulate principles and methods for the valuation of railroad property and other public utilities." The Committee may find justification in making a report on "valuation for the purpose of rate-making," in its statement that "Valuations of the property of public service corporations may be made for a variety of purposes," and its report on this particular variety may then be considered as only one section of a general report, the other sections of which are to be furnished later.

The Committee goes on to say:

"It is, of course, true that the principles and methods of valuation for rate-making are in most respects the same as those involved in the valuation for other purposes."

Granting this truth, if the Committee had confined itself to a general report, it would have avoided the criticism of engineers, specially interested in rate-making, who differ with the Committee in some of its conclusions. The Committee's report is on "Valuation of Railroad Property and Other Public Utilities for the Purpose of Rate-Making." The immediate and most important application of the principle of valuation for the purpose of rate-making is found in the case of the

Mr. Bates. railways; but the report seems to be based largely on experience with water-works and public utilities other than railways. The report would have been more acceptable if its treatment were limited to local utilities which constitute monopolies in their own limited territory, such as water-works, gas plants, electric properties, etc. As a matter of fact, the report is based on the condition of, and legal decisions relating to, such utilities; but it also covers in a desultory way questions of valuation of railway property, and really lumps the whole proposition to include them. It attempts to apply the same principles to circumstances and conditions which are entirely unlike.

The rates to be charged for gas, water, telephone service, local transportation, etc., in Chicago, are not governed or influenced by the rates permitted in St. Louis or Milwaukee. The same principles may or should govern; but there are no competitive influences or conditions. For the regulation of those local utilities which are non-competitive, or competitive only in a very limited way, principles of valuation may be formulated which will be approximately fair and just; but the situation with railways is entirely different. Whatever advantages may come from railway valuation, the proper fixing of rates is not one of them.

The Government valuation of railways is not declared to be for purposes of rate-making. Authority to make the valuation, after repeated requests by it, was granted to the Interstate Commerce Commission which wanted it for whatever value of any kind there might be in the information. The Commission has repeatedly declared that rates cannot be based on valuation, although it may be one of many elements to be considered under certain conditions. It would be just as fair to assume that the valuation is being made for the purpose of Government purchase and ownership, as to assume that it is for the impracticable purpose of determining fair rates.

The impossibility of basing railway rates on the plan of fair interest return, or even profit, on the value of the property, is so patent that it ought not to require any re-statement. The whole railway system is competitive, and the Government, just now, is bent on breaking it up into still smaller competitive units. Between certain terminal points there are half a dozen or more lines with different values, whatever principle of valuation is used in determining them. All must charge the same rate; and no road can, of itself, fix its own rates.

The existing rate structure is the product of commercial, competitive forces, and as a whole is found, even by regulating bodies, to be about right. It needs constant adjustment to meet changing conditions and to insure reasonable equality. Following a world-wide change in wages and all costs, the entire rate structure may need to be increased horizontally—as is the case at the present time—but no road or system, even if legally permitted, could raise a rate by itself,

because its traffic would immediately desert it and go to other lines. This is not true of local public utilities, because there is practically no other available supply of service for the public. Mr. Bates.

If all the railways were in one system and ownership, rates might possibly be predicated on value of investment, and changing costs be met by changing rates, so as to insure an average net earning. The strong lines would then support the weak; but, with diversified ownership and the demands of all parts of the country for adequate transportation facilities, nothing of the kind can be done, and speculation regarding it is useless and misleading.

The Committee should either have differentiated clearly and omitted railways from its discussion, or should have omitted entirely that part of its report relating to "purpose of rate-making."

The questions involved in the valuation of public utilities for the purpose of rate-making are of such importance and of such direct interest to the members of the Society that the fullest discussion of them should be encouraged, and final conclusions should not be adopted until discussions are exhausted. With constant changes occurring, the time when discussions will be completed may never arrive, and final conclusions may be indefinitely postponed. Such a result ought not to be regretted. We must act according to our best present information, and continually move on to better conditions.

It should not be assumed that the questions to be solved in valuing public utilities for rate-making are wholly of an engineering nature. In fact, the engineer has only a limited function in their solution. The valuation of public utilities for the purpose of rate-making calls not only for engineering knowledge, but as well for the knowledge possessed by lawyers, by financiers, by real estate experts, by business men, and by accountants, all of whom should state the cases, from their points of view, before an attempt is made to formulate definite conclusions.

The work of valuing railway property, recently inaugurated by the Interstate Commerce Commission, will be done under certain disadvantages, some of which are:

A large organization, not trained to the work at hand, but gathered at one time from such material as is available, with the certainty that the railways will retain in their own service the experts who are fitted by experience for this work; the demand for results from this organization necessitating conclusions before this new occupation is capable of producing correct conclusions based on knowledge. In other words, results will be published before the business is learned, and conclusions will be to a greater or less extent untrustworthy.

There will be the interpolation of politics, directly and indirectly, into a fight, for power on one side and for existence on the other side.

Mr. Bates. It is claimed by railway officials that the total value of railway property in the United States is $15 000 000 000, more or less. It is not true, as might be inferred from the assertions of politicians and of sensational newspapers, that this property is owned by a few magnates whose only concern is to rob the public. The railways, collectively, are a useful and valuable asset of the whole nation. They are owned by the holders of their stocks and bonds, mostly innocent investors, including men and women who have bought the securities with the accumulation of their savings, and children and helpless people are represented by the investment of trust funds in these securities.

The American Society of Civil Engineers cannot afford to place its endorsement on a report which may be used to increase or decrease the returns to the owners of these railway properties. If the report was absolutely conclusive, the Society could with propriety stamp it with approval, establishing the truth, without regard to its effect on any interest; but this is not the case. The principles claimed and the conclusions drawn will be disputed by a large number of the members of the Society. The Committee itself does not consider this report as final, for, in foot-notes at the beginning of the report and on page 74 it reserves the right to add to the report before its final publication.

This report should be received by the Society without any endorsement, good or bad. It is carefully prepared, and contains much valuable information. It will be interesting and useful when published in the *Transactions,* but it should not be assumed to be an expression of the Society as a body. It should simply be received with thanks. The reception and publication of such a report is an honor to its writers. The Society is careful to publish the statement in its *Transactions* that it "is not responsible for any statement made or opinion expressed in its publications." This statement, which applies to every paper published by the Society, should have equal application to the report under consideration, and should apply with special effect to it, for the reason that there will be great diversity of opinion regarding its contents. Even if it were considered necessary for the Society to take action regarding the approval of the report, it would not be right for such action to be taken at the Annual Meeting, which is attended by a comparatively small number of members. If a division is sought on the question of approving the report, it should be submitted to the whole voting membership of the Society for such action.

The writer has received a letter from Mr. Jared How, an attorney-at-law in San Francisco, criticizing the Committee's treatment of the subject of depreciation, and, being in full accord with Mr. How's criticisms, he offers (with the consent of Mr. How obtained by telegraph) this to the Society for its information, and as a contribution by Mr. How to the discussion.

JARED HOW, ESQ.* (by letter).—The writer's attention has been called to an advance copy of the report of the Special Committee of the American Society of Civil Engineers appointed to "formulate principles and methods for the valuation of railroad property and other public utilities" which is designed apparently to be presented to the Annual Meeting of the Society. The writer has been allowed only a casual inspection of the report, and was advised, of course, that what he learned from it was to be held in confidence. Nevertheless he feels at liberty to mention it and criticize it. Mr. How.

The writer paid no attention to the contents of the report excepting so far as it deals with the subject of depreciation. He had too little time to read the subject matter of that topic with care; but is impressed with the idea that the Committee takes an unfortunate attitude toward it.

The basal principle of the relation between a public utility, a steam railroad, for example, and the public, is that of contract. The right to construct and operate a railroad is not a general right which appertains to every citizen as such. It is a right obtained only through the grant of a franchise from the sovereign power. The grant and acceptance of the franchise create a contract, the consideration of which on the part of the public is executed by the grant, and on the part of the grantee is executory and rests in the obligation to exercise the franchise in the manner contemplated by the grant. So a railroad company, having accepted a grant of a right to construct and operate a railway as a common carrier, is under contractual obligation to operate it as such and to furnish adequate and safe facilities for the conduct of all business which shall come to it. In other words, it is under obligation to keep its road and equipment up to 100% condition for efficiency, and to operate it up to that standard so far as the needs of the public require service. The manifest difference between the relation with the public of a warehouseman or an innkeeper, for example, and the owner of a railway, is that while the use of the property of each is of such character that the public has an interest in it and therefore may regulate the exercise of the use for its own protection, the warehouseman and innkeeper are under no obligation to continue their properties in the public use, and the owner of the railway is under such obligation. Furthermore, the warehouseman and innkeeper are not under obligation to furnish ample accommodations to serve all who shall apply to them. Each of them is justified in refusing to accept offered patronage, on the ground that he lacks facilities to care for it. The owner of the railroad property is not thus justified. If his facilities are insufficient, he must provide himself with more. The only pos-

* Attorney-at-Law, San Francisco, Cal.

Mr. How. sible ground for the distinction is that the warehouseman and innkeeper have no contract relation with the public, and the owner of the railroad has such a relation; and his obligation growing out of that relation is to provide himself with such property and to operate it, when acquired, in such manner as shall be, not only inoppressive to the public, but, as well, adequate for the reasonable necessities of the public for the contemplated service for which his franchise was granted to him by the sovereign.

The fact that property of a railroad company is devoted to the public use does not make it public property. It is still private property, although it must be used for the service of the public; and, as such private property, it is under the protection of the provision of the Constitution of the United States that no State shall deprive any person of property without due process of law—and by "property" is meant, of course, the profitable use of property, as well as the title to it and dominion over it. Whether an owner of property is deprived of its profitable use seems to the writer to be clearly a question of fact to be determined by the application of the principles of economics, and not a question of law. The writer, therefore, comes to the consideration of the question of depreciation of railroad properties—the precise question being whether confinement of a railroad owner, whose property is maintained in 100% condition of efficiency, to a return on its depreciated value predicated on its cost newly produced, and not on its efficiency as an instrument, is, economically considered, a partial deprivation of the valuable use of his property.

The writer has before him, professionally, an estimate, made by an engineer of a State, of the present value of a line of railway. This railway was constructed as one task within a period of about 2 years, and cost about $5 000 000. The property was operated for some special traffic during construction. The valuation was made as of a date about 20 months after the road was turned over to the operating department. The State engineer finds its original cost and its reproduction cost; and then he estimates the "condition per cent." of each item of physical property, and averages them for the total, with the result that he estimates the present value—the value on which the owner is entitled to a return without interference by the State—as 95% of the amount of present cost of the property. If the property actually and reasonably cost $5 000 000, 2 years before this valuation was made, and could reasonably be reproduced at the time of the valuation for not less than that amount, as might well be assumed, then, if the owner shall be limited at the time of the valuation to a return on only $4 750 000, it seems to be clear enough that somewhere he has wholly lost, or has at least lost the valuable use of, $250 000. There is no force in the suggestion that he is entitled

to earn and lay aside this $250 000 during this period of 2 years in which the depreciation is alleged to have accrued. Even if he shall be able to earn it, he may not consider it as a return of a part of his investment and devote it to his private use as such. He is under exact obligation to use it in replacing the depreciating articles when they shall have depreciated to the point of ineffectiveness. In other words, because the proper maintenance of the property is the very first charge against the earnings of a railroad company, and because the property must be maintained ordinarily in perpetuity, money properly necessary for replacement of portions of the property when they shall have depreciated into inefficiency, and reserved by the company for that purpose, may just as well be said to be devoted to the public use as may the physical property of the company actually used in operation. It may, perhaps, be invested by the company until the necessity for its use arises, but if the investment proves unfortunate and the money is lost, it must be replaced, and, apparently, at the expense of dividends which otherwise might be properly payable; and, if the depreciation fund is invested and produces income, of course that income ought to be included in the general income of the company on the net of which the reasonableness of its rate of return is to be figured. Therefore, if the physical property is to be valued on a depreciated basis, obviously the depreciation fund necessarily segregated for the purpose of covering depreciation should also be valued, and a fair return should be allowed on both. If one builds a line of railroad 150 miles long, the cross-ties may all be laid in one year and may cost $325 000. Assuming the life of the ties to be 8 years, the railroad, in the eighth year of its existence, is none the less valuable to the public and none the less valuable to the owner, if he has on hand, and applicable to no other purpose, money sufficient to replace the cross-ties at the end of the year. Why then should he be limited to a return on the value of the property, less cross-ties, unless, indeed, he shall have added to that lessened value the amount of money which he shall have laid aside as necessary for furnishing new cross-ties? The writer cannot avoid the conclusion that if the owner is thus limited he is deprived of the valuable use of a portion of his property devoted to the public service. Mr. How.

This is not much of an argument. The writer has not time to be as convincing as he is sure the soundness of his views on the question would permit. He writes because he thinks it will be calamitous if the Society goes on record in favor of economic practice, which seems to be utterly iconoclastic, and without a dissent.

W. H. Courtenay, M. Am. Soc. C. E. (by letter).—Under the head of "Engineering", occurs the following statement: Mr. Courtenay.

Mr. Courtenay. "On railroads it is commonly estimated that the engineering cost will amount to 5% of the physical valuation of the property, exclusive of overhead charges."

The Committee proceeds to give certain specific examples of the engineering cost of other public service work, but does not give any specific examples of railroad work other than the subways in Boston and the Pennsylvania Railroad tunnels, both of which are out of the ordinary.

If the Committee has other data as to the ratio of the cost of engineering to that of construction on railroads, it would be desirable information.

Mr. Hendricks. V. K. Hendricks, M. Am. Soc. C. E. (by letter).—This report covers the question of valuation in an excellent manner, getting down to the fundamental principles, and the writer considers the method proposed good for appraising property with a view of obtaining an equitable sale price. However, it requires a change in the system of accounting now in use on railroads, and as the same results can be obtained for rate-making purposes by using the valuation new, without changing the accounting system, it seems preferable to omit depreciation for rate-making purposes. It would be quite objectionable to change the system of accounting on all railroads.

In addition to this objection, it is believed that the use of the valuation new would cause less danger of errors through determination of the exact status of depreciation on each property, and would thus produce more uniform results, which would be more equitable to new and old properties alike.

By using the valuation new, it would be necessary, for the party calculating a rate determination, to consider each long-lived item separately, using the adopted percentage (which is practically uniform) corresponding to Columns 6 or 7 of the table showing the "Equal Annual Payment" method, and taking no account of the amount actually charged to operating expenses for the maintenance of such items. Short-lived items, which could not conveniently be given such special consideration, would be considered through the operating charges for maintaining such items, in the same manner as proposed by the Committee. By eliminating the actual depreciation for each individual property, the work of preparing the valuation, of all railroads, for instance, would be materially reduced and also the work of the "rate-maker", in that his calculations on long-lived items would be made from tables based on the expected life of the various items, and would be uniform for all similar properties, instead of requiring a different set of calculations for each year of any one property's life. This method accepts the theory contained in the Committee's report, and merely provides for a different method of application.

The writer, therefore, recommends that, for rate-making purposes, the Society endorse the use of the valuation without any allowance for depreciation. This is an arbitrary ruling, but attention is called to the fact that in its report the Committee has made an arbitrary ruling in distinguishing between the method of handling long-lived items (tentatively assumed as having a life of 5 years or longer) and short-lived items, such as lining and surfacing of track, etc., the maintenance of which is just as much a restoration after depreciation as is the maintenance of the long-lived items. Mr. Hendricks.

Under the heading, "Application of Equal Annual Payment Method", it is stated that "the placing of additional ballast on a railway roadbed" should be charged to current expenses. The writer thinks this should read "the replacing of ballast on a railway roadbed", as new and additional ballast should certainly be charged against the capital account.

E. W. James, Assoc. M. Am. Soc. C. E. (by letter).—In the large mass of expository and argumentative material produced during the last three years on the matter of valuation for rate-making purposes, only a small percentage can be dignified as contributions to the literature of the subject. The most prevalent fact—so common, indeed, as to be usual—is the failure of writers on the subject to establish any fundamental principles; and next, to confuse, sometimes in unexplainably simple ways, the elemental economic truths. The early days of many public service corporations display a tangle of promotion, corruption, lease, combination, purchase, and reorganization. Original records in many cases are lost. As a result, the discussion of physical valuation has been largely confined to consideration of the ultimate, technical details of some practicable method, to the neglect of important underlying principles. Valuation has been usually argued from the point of view of the material interests of the disputants. It has been only within the past two or three years that really serious conclusions have been arrived at. The report of the Committee, as presented, gives something lucid, fundamental, and unbiased, on which to hang discussion. The Committee deserves much credit for covering the subject in such detail, and at the same time in recognizing the basis in principle so essential to the whole matter. The report is a most valuable contribution. Mr. James.

At the same time, it was to be hoped that somewhat fuller exposition would profitably be made with regard to the economic and social principles that most certainly form the substructure of the whole valuation edifice. The Committee recognizes this idea when it states that:

"The primary requirement in the valuation of a public service property is that justice shall be done both to the owner of the property and to the public."

Mr. James. This implies "fair rates" adjusted to produce "fair returns"; but it is obviously necessary to go back of this statement. It would not be just to close our eyes to past conditions, in any of their aspects, and rest our justice on present contingencies alone. Some papers have discussed at length the violation to property rights that would inevitably flow from any valuation that did not produce a fair return on the money investment of the present holders of stock. Such a valuation, of course, would do magnificent justice to the present stockholder; but, unfortunately, it would be saddling the public with a burden of fictitious values of all kinds for an indefinite future. This extreme case shows that the actual investment of the present stockholder is dangerously subject to sharp contraction in value; that such investments, heretofore seemingly sound, are to be a source of loss; that the investing public, like the proletariat buyer and more modest wage-earner, cannot look to the public for guaranty of all their past transactions. The investors took chances, naturally in the hope of adequate gains. It would be highly unreasonable, now, to remove all or nearly all the element of risk from such investments and assure to the stockholder for all the future a fair return on all the paper values he holds. The fact must be frankly recognized that someone is going to lose, and perhaps lose a great deal, in any adjustment of conditions flowing from rate-regulation on a valuation basis. It is inevitable that an older ground of justice than present conditions afford be sought and accepted as the really equitable and effective one.

In support of this, consider for a moment the recent events in connection with the passing of a dividend by the New York, New Haven and Hartford Railroad, and the insolvency of the St. Louis and San Francisco. As a result, perhaps, of the intricate financial maneuvering indulged in by the managers of these roads, the stockholders have had to accept some investments that are likely to turn out bad. They are going to lose some money, perhaps lose only paper values. Maybe they have been mulcted by trusted officials. In any case, however, whatever degree of truth there may be in any of these suppositions, certainly the general public should not be called on to relieve the stockholder of loss, or guarantee him for the future a fair return on funds out of which, frankly speaking, he has been cheated. So, it must be accepted that present justice cannot be done. Present holders cannot be entirely protected. It would be as just to have a public guaranty that all past real estate ventures by individuals in a given community shall result in gain, or at least in no loss. What the community should do in the future in this matter, however, is quite another affair.

This point of view probably makes the injustice loom large; but is the degree of probable injustice as great as it appears? Considering the holdings of stock in such roads as the Louisville and Nashville,

the Pennsylvania, and the New York, New Haven and Hartford, it is probable that not more than a very small percentage of all public utility stocks that may be affected with loss as a result of rate-regulation on a fair plan, are held as *bona fide* investments by the people who, for instance, deposit in savings banks. A large mass of these holdings are held by direct, though in some cases now temporally remote, virtue of past manipulations. The holders took large risks and knew it. Their holdings do not represent vested interests of a sort that the public is bound for all time to hold sacred. In the past these holders have made huge gains; often their predecessors made huge gains. The profits they hoped for when they assumed their original risks have been forthcoming. Now, it appears likely that there will be a period of change, a time of reaction, perhaps some loss; but, in essence, the injustice will be of a blameless sort. In some cases innocent investors will lose, some homes will be impoverished, some lives saddened; but such painful facts are a part of all economic change, for good or bad, and are everywhere recognized in the common-sense fact that all investments cannot be successful. "Lambs" have been losing for years on the stock exchanges of New York, Chicago, San Francisco, and New Orleans. "Suckers" have invested millions of hard-earned, lonely dollars in wild-cat schemes of all kinds that appeared to them to be just as certain of returning substantial profits as the Pennsylvania or the New York, New Haven and Hartford Railroads. It is pitiable, but it is true. Mr. James.

A deeper-founded justice must be sought whereon to base an arrangement for fixing rates. It would, the writer ventures to suggest, have been profitable and illuminating had the Committee included in its report a more extended examination of the principles that are sure to be recognized sooner or later. Especially does this aspect of the general question appear inviting in connection with the most wise suggestion of the Committee, that:

"It would be entirely just and equitable, and it is highly desirable to provide by law that future public service properties should be valued on the basis of their actual reasonable existing investment."

The great value of this suggestion cannot be too soon recognized and acted on. No problems of any complexity would exist regarding such valuations, for the method of appraisal would be stated in advance. The experience that German municipalities have commonly had indicates that American concerns are quite ready and able to enter the field of furnishing public utilities when the investment is closely scrutinized by State or municipal authorities, when the rates are fixed, and when all net receipts above a certain permissible income on actual, reasonable investment are returnable to the State or city.*

* "Municipal Government in Continental Europe," Albert Shaw, pp. 52, 350, *et seq.*

Mr. James. These conditions are considered satisfactory, and are met by American investors abroad; it is reasonably assumable that they would be acceptable to American investors at home. The sooner the conditions and limitations affecting future public utility valuations are prescribed, the better temper and security will be enjoyed by that part of the business world involved.

This recommendation of the Committee regarding the provision for making valuations of new utility properties, or for revising old valuations henceforth, indicates that, however much the Committee may believe in cost-of-reproduction-new as the most satisfactory method for making present-day valuations of old properties, it, nevertheless, is of opinion that actual reasonable cost to date of the property useful and usable in the public service would be the most just and satisfactory basis for valuation purposes. If the Committee recommends it for the future, it must at least accept its principle as sound; and therefore failure to use it for the valuation of old properties flows from considerations of expediency or compromise.

In spite of the fact that the Committee has chosen not to emphasize the fundamental principles, or to explain at greater length its opinions in this regard, it nevertheless arrives at generally sound conclusions relative to many of the matters which have long been in dispute.

There are, however, such fundamental principles, and although a general discussion of them belongs rather to the field of the economist, and may be considered by some as much too academic in nature to be of really practical value, still there is no denying that the closer we adhere to the course prescribed by sound and true principles, the more certain it is that substantial justice will be done to the greatest number. When justice is referred to as entering into the purposes or results of physical valuation for rate-making, an ethical principle is assumed and admitted, though perhaps unconsciously. This is an aspect of valuation that has not received the emphasis it deserves. A study of the economic and social phases of the subject may be made much more illuminating that it has been. It appears that such a study would provide a solvent for several of the problems now in dispute among writers of undoubted ability and experience. In the first place, the public attention has not been directed toward corporation control because it had nothing else to busy itself with. The movement toward physical valuation is not an erratic one. The public's point of view has of late years been changing. People are seeing that they have an interest in public service corporations different from that concerned merely with securing good gas, steady current, accommodating trolley service, or pure water. The new laws providing for physical valuation as a basis for determining rates for the use of public utility products or services are an evidence of this change of attitude.

That the public, the user, has been slow to recognize its real interest

in public service corporations does not in the least lessen the rightfulness of that interest, or its substantial basis in fact. A savage has little or no ethical life; for, although an ethical principle may be absolute, *per se,* the strength of that principle applied depends on contributory circumstances of a nature to compel its recognition. Circumstances of corporation growth, the misuse of the power flowing from privileges granted to individuals by public official bodies, have at last forced the user of the public utilities to recognize his ethical right to consideration. Mr. James.

The interest of the user in the public service company is, of course, first of all, material. The company furnishes some service necessary to health or to the reasonable comforts of modern life, or something necessary to business. These services cannot generally be secured except from the public service corporation. For this reason the corporation is looked upon rightly enough as a quasi-public organization. It is no longer a private affair. In addition to this material interest, arising in the need of the community for fresh and pure water, light, heat, and transportation, there is the other, deeper consideration spoken of above. This flows from the material interest, and is best described as an ethical interest. It is not merely by equity or expediency that the user has rights in public service utilities. It is because he puts something into the business. What he puts in is, from the viewpoint of the user, fixed capital; he cannot withdraw it, and heretofore has had little or nothing to say regarding the management of it. Much of this fixed capital, supplied by the user of the public service utilities, is in the form of franchise privileges granted by the user, through his representatives in the local legislative body, for a period of years or in perpetuity. It is just as surely a vested interest as the money of the bond holder which has laid rails, strung wires, dug ditches, or erected pumping stations and gasometers. The fact that the consumer turns the management of his interest in the business over to the other party, together with the fact that when once these contributions are made they cannot practically be withdrawn, establishes the right of the user to fair treatment at the hands of the public utility company. Still stronger appears the user's right to fair treatment when it is considered that he is, in the very nature of the case, the residual investor. That is, under any plan to establish such rates as will provide a "fair return" to the money investor, the user must hold himself ready at any time to meet all demands for financial support involved in the operations of the utility. The user may not provide additional money capital in the early years of development, and may never indeed do so; but, if such becomes necessary, the user must, we admit, assume the burden of interest charges created by the required borrowing. Mutual obligations between the user and the producer follow naturally as a result of mutual interests.

Mr. James. These obligations are as binding on one as on the other. Responsibility for fair treatment has in the past rested with the producer exclusively, because he has occupied the position of control. Unfortunately, the user has not been convinced, by the treatment that he has received, that his rights have had the recognition to which they are entitled.

A very interesting account of some of the increases in capitalization which have been made by railroads during the past 40 years, appeared recently.* Though the citations are doubtless extreme, their number is so large and the sums involved so enormous, that, had no other instances occurred, there is sufficient ground for the introduction of sudden and drastic control of the financial management of the railroads. At the same time, much praise is due to the operative management for their successful efforts to pay dividends on no less than $784 000 000 for which not a cent has been added to the value or earning capacity of the railroads. After extended consideration, Professor F. W. Taussig declares, "it is doubtful whether the whole mechanism of irregular and swollen capitalization was at any time necessary or wise."†

Nevertheless, it has been the user who has had to pay the bill, and, as a quasi-partner in the concern, it has obviously not been just to him to add paper capital which shall stand for all time and on which he must pay dividends.

As a result of these conditions, the present movement toward some degree of financial supervision has resulted. It is time that the relative obligations of both the users and the producers be understood. The railroads are by no means the only public service corporations involved, but they are by far the most important, both in relative value and in economic status. Further, they are to-day so generally the only corporations of a semi-public character whose life has been long enough to become seriously complex or whose records have been seriously impaired by neglect or loss, that a solution of the question of regulation as applied to railroads will, if satisfactory, serve as a solution of the more general cases.

The fact that the railroads hold so prominent a place among public service corporations, the added fact that among such corporations they stand almost alone in doing an interstate business, and consequently are the most prominent subjects for the exercise of control by the Interstate Commerce Commission, gives them a place in the discussion that cannot be deprecated. In accord with this condition, every development of practicable methods of valuation has held the railroad in chiefest consideration. It is doubtless due to conditions existing in the accounting offices of the railroads, that the general

* "The Plight of the Railroads," W. J. Lauck, *North American Review*, January, 1914.

† "Principles of Economics," Vol. II, Chapter LXI.

opinion of those who have had to give consideration to these matters is strongly in favor of cost-of-reproduction-new as a basis of valuation for rate-making. Mr. James

As the writer has pointed out previously, the Committee recognizes the weight of opinion in favor of such a method, but at the same time supports the view that in the future the actual reasonable existing investment should alone be accepted as a basis of the same valuation. Apparently, the Committee has adopted cost-of-reproduction-new instead of original-cost-to-date merely as a matter of expediency, and in a spirit of compromise toward the confusion of conditions existing to-day among railroad records and old methods of accounting. In the writer's opinion, it would be much fairer and much nearer conditions of justice if the basis of valuation were made original-cost-to-date, just as nearly as it could be arrived at. In many cases this cost could be arrived at but remotely; nevertheless, it could be found in many cases as accurately as cost-of-reproduction-new. Nothing is gained in compromising on a point of principle. Rather, first, establish the principle. If it is advocated as good for all the future, it must be good enough for all the past. If the principle is established as applicable to the past, the law can then legalize it for the future. Indeed, the difficulties of uniformly applying a set of principles to conditions that the past hands down to our time are an argument for statutory action. In the presence of these difficulties it will be necessary to adjust methods of application. This can be done. It is much better to adjust the application in particular cases than to sacrifice a principle. If attention is given to that principle suggested above, that the user has, by virtue of existing conditions, an ethical right in the property of any public service corporation that acts under a franchise or to which government has transferred at any time the right of eminent domain, it will be seen that there is only one just basis of valuation. That is cost-to-date, actual, reasonable investment. The investment does not necessarily include all the money spent in creating the utility. It is no part of the user's duty to secure to the investor a return on funds spent unwisely, unnecessarily, or in any improper way.

Of course, those who support the position taken by the Committee cite the fact that complete records for cost-to-date do not exist in numerous cases. Such contention is doubtless true; but, admit this, adopt the cost-new method, and at once uncertain conditions are met, and details are prescribed that are really an effort to approximate cost-to-date. Assuming that the records are lost or imperfect, what is the essential difference between an appraisal made in an attempt to approximate original cost-to-date and one based on cost-of-reproduction-new in which "the original conditions should control," and in which costs "should be based upon physical conditions existing

Mr. James. at the time the various portions of the property were built?"* The Committee recognizes again that the original cost is the one to be approximated, when, in its summary† it suggests that "average prices prevailing during a period of from five to ten years prior to the date of valuation should be used." In this way the present and the past are made to overlap, and just when this shall be done and to what extent is itself a nice question. For who shall say when the old becomes old, or how much of the new remains new? After all, is not the question but little more than one of definition? Does not the Committee resort to the method it considers less desirable in order to correct too faulty results produced by the method it considers best? Is not the valuation suggested by the Committee, under any name one pleases, after all, the actual reasonable investment represented by the properties under consideration, as nearly as it can be found? It appears to be, so it should be fairly met. In the ethical principle suggested above there is sufficient cause for adopting such a basis of valuation.

Furthermore, this ethical principle is a solvent of several problems that excellent authorities now disagree about. This disagreement, as so frequently is the case, arises in all probability because there is no common ground of principle on which to rest the discussion. If we recognize the ethical right of the user in the public service corporations, we have at once the principle we need. Consider for instance the matter of land valuation. The Committee, after writing off depreciation, sees that the appreciation presents a perplexing question, and states:

"The Courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value."

Would the Committee hold that the public should receive any allowance due to a settled, stable grade, as contrasted with a new, green embankment? Consider this matter of land rather in the light of the ethical principle involved. The difference of opinion is really over the unearned increment. The usual and principal argument to support the inclusion of the unearned increment is that the public service corporation should not be denied what every land holder is allowed. Professor Taussig adopts this same position,‡ but points the way to an answer. Of course, if a valuation were being made for sale, taxation, or rental purposes, the inclusion of the increment would be right. If a utilities corporation chooses to rent or sell, it doubtless

* "Committee's Report," pp. 18, 15.

† "Committee's Report," Section 9, p. 71.

‡ *Supra.* Vol. II, Chapter LXI.

is entitled to take advantage of the unearned increment, if it can find a purchaser on such terms; but the unearned increment of the land does not represent an investment of the producer but of the user. The increment arises from the development of the land contributory to the railroad. It is greatest in sections where the development is greatest. It is in essence a contribution of the community. The community is nothing more nor less than the user in an aggregated form, and as we have seen, the user is a co-investor with the producer in public utility properties. Obviously, no part of the return on the user's investment should be included in a "fair return" to the producer. It follows, of course, that lands donated, whether by a person, by a commercial body, or by government, should not appear in the valuation at all.* Mr. James.

The Committee itself may have had some misgiving with regard to the decision to include unearned increment. For the existence of a doubt is suggested in the expression used: "that if it be admitted that the [acquired] property is to share in the increase, etc."

The recognition of an underlying principle would not have led the Committee to decide that donations of either money or land by a community should be capitalized and made a basis of dividends to be paid by the community.

It is, of course, unnecessary to state that all legitimate expenses incurred in connection with the acquisition of land, all damage, transfer, and severance costs, should be included as a part of the cost of the land to the corporation.

Likewise, all necessary engineering charges, legal expenses, interest during construction, just exactly as during any later time, should all be allowed, and the recognition of an underlying ethical principle supports this view. Why, then, does the Committee say that "the amount of the overhead charges applicable to any property should be determined on the basis of the prices prevailing at or near the time of the valuation, but under conditions existing when the plant was created?" It will be as difficult to approximate the conditions existing twenty years ago as to approximate prices paid twenty years ago, and probably much more so. The aim should be to approximate both conditions and prices at the time of acquisition, and, indeed, the Committee probably sees this when it says, "the proper purpose of a cost of reproduction estimate of the value of real estate is to ascertain the cost of acquiring such real estate under the conditions existing at the time of its acquisition."

The Committee goes on to recite some special values attaching to land. Undoubtedly, if such values were recognized at the time of acquisition, and if a corporation was forced to pay a price for them,

* Similarly, bonds guaranteed by counties, a procedure not at all uncommon in the Southern States in railroad promotion, have no place in a valuation for rate-making purposes.

Mr. James. such values should continue to be recognized in stating or approximating the original cost; but any law regulating valuations of future properties should rigorously exclude such values. For, obviously, such special values do not attach to the land; they come into existence in the land only when the utility enters upon it. The Committee's stand in this matter is undoubtedly right.

When the question of excess or deficiency of past earnings is considered, our attention is again directed to the advantage of emphasizing some principles that should control in establishing methods of valuation for rate-making. The Committee suggests that deficits occurring in the development period, if covered by the producers, should be capitalized, and this suggestion is well founded; but, if such deficits are made a basis of future dividends to the corporation, why, on the other hand, should not past excess earnings go to offset such development deficits? Surely, if the earlier records, covering transactions and financial conditions during the first years of existence, are to be considered adequate as a basis for determining development costs, then subsequent records, for years when organization has improved and crystallized, may properly and reasonably be considered adequate to determine the excess earnings.

Two very interesting matters are presented for consideration by the Committee relative to efficiency of operation and special advantages in site or conditions of production. No definite suggestions are advanced, however, with the view of introducing these elements into a physical valuation, beyond a general recommendation that they receive due consideration at the hands of the Courts and of the regulatory commissions. The second of these questions is one of economic rent, and has all the difficulties attaching to such. It involves the really serious question as to what poorer, less efficient organizations are to do, if rates are adjusted for highly efficient competitors in the same territory. In the nature of things, the poorer road would lose business unless it meets the competition. On the other hand, there is a serious situation created if rates are adjusted for the efficient road so that when applied to the inefficient road adequate revenues will be produced to operate and pay returns. There is a solution, but no one known to the writer has had the temerity to suggest it in this connection. The solution is to adjust rates to make the less efficient or less favorably located road pay the minimum return. Adopt a sliding scale of fair returns and permit the stronger road to charge the same rates as the weaker and pay to the State, in form of taxes, all but the necessary revenues to operate and pay a fair return according to the sliding scale. The higher permissible rate would then save the weaker road, and be sought by the stronger, and the sliding scale would preserve the required incentive to increased efficiency in the case of both, and prevent excess earnings.

Obviously, such matters are due for serious consideration at a much later time, when regulatory methods are more advanced than at present; but such problems are certain to demand solution sooner or later. Those analysts of the present situation who are most disinterested, who see the tendencies of our time writ large and growing imperatively to demand a stronger control, do not hesitate to declare that the success of democratic government depends on greater discipline, more restraint, and fairer adjudication in all the relations of large-scale production with the general public.* Such control is not impossible. It exists in some nations of Europe. Such discipline as may be necessary does not necessarily destroy the incentive to save and invest, for a much severer discipline in Germany and Austria than will be developed here in many years does not do so. Mr. James.

The great question left is that of a fairer adjudication. This demands action in our time, change from present conditions; and all change hurts. The railroads constitute undoubtedly the greatest single development in the economic life of the western hemisphere. They are largely responsible for the general industrial development of the United States, and have played no small part in producing the seriously perplexing questions of regulation of public service companies. It would be a national disaster to hamper the railroads by a maze of restrictions regarding their operation, and great care must be exercised in this respect. Some States have already, doubtless, gone too far. Railroad operating managers are more expert in such matters than legislators; but control and supervision of railroad finances must be accepted as imperative. Such control is eventually to be continuous, and, in moving toward it, the chief consideration should be given to the principles underlying the elemental relations of the public and the corporations. Past wrongs must, so far as possible, be righted; but that no complete adjustment is possible is generally recognized. The greatest good can no doubt be effected by sane constructive legislation providing for control. On the success of this movement rests no inconsiderable part of our future national prosperity and progress. Engineers, as managers and as men of affairs, are wonderfully well equipped to handle these problems, and can hardly give them too much attention.

W. W. K. Sparrow, Assoc. M. Am. Soc. C. E. (by letter).—This report is most complete and exhaustive, and is certainly a very valuable addition to the literature on this subject. The Committee deserves the thanks, not only of the Society, but of all persons interested in the valuation of public utilities, for the time and thought it must have given to the preparation of such a report, covering as it does prac- Mr. Sparrow.

* "Principles of Economics," F. W. Taussig, Vol. II, Chapters LX to LXIII.

Mr. Sparrow. tically all the basic and fundamental principles underlying the valuation of public service corporations.

The writer has not had the time to digest the whole report thoroughly, and will confine himself at present to the question of development expenses and land values.

The Committee defines the term "Development Expenses" as "the investment necessary to put the plant into successful operation and to create revenues that justify its construction," and prefers to use this term to that of "Going Value." The writer fully agrees with the Committee that the term "Going Value" has been given so many interpretations by the Courts and others that it is pretty generally misunderstood, and its use is inadvisable.

The Committee is of the opinion that, except in the case of a losing venture, the development expenses, as already defined, should be included in the cost of the property. The writer has given some thought and study to this question, and is not in agreement with this recommendation. It is understood at the outset that the report of the Committee, and therefore any discussion on it, relates to valuation for the purpose of rate-making only.

Development expenses may be divided into two classes:

(1) Expenditures made to create or attach business;
(2) Loss of dividends, and loss due to the earnings being insufficient to pay operating expenses.

The principle involved in the Committee's recommendation is that such expenditures as are classified in the first division and such losses as are classified in the second division constitute a property investment in the business, and that as such the general public is required to pay a return on this investment, or, in other words, that the company is entitled to capitalize such expenditures and losses on the same ground that it is entitled to capitalize interest paid on capital during construction.

Does any live well-managed property ever reach the point in its life when it is not necessary for it to expend large sums in developing and acquiring new business? On the contrary, is it not true that the larger and more prosperous a business becomes, the larger these expenditures become? Has any one ever heard of a company charging such expenditures to capital account and asking a long-suffering public to pay a return on such investment? Such expenditures are universally recognized and treated as a part of the operating expenses, and the writer can conceive of no good or logical reason for treating such expenditures, made at the beginning of the life of a business, in any different manner than that in which they would be treated if made during any other period of the life of the business. They are operating expenses

first, last, and all the time. Recognizing this fundamental principle, the next point to consider is whether a company is entitled, as a legal right or in reasonable equity, to capitalize deficits in its operating expenses incurred during the first few years of its life. If such deficits are capitalized and treated as part of the investment of the company on which the public must pay a reasonable return, it logically follows that the return for depreciation, which is based on the total investment, will be increased in proportion to the amount of these losses, and, as a result, the public will in time, not only amortize such losses, but continue to pay a return on them. Mr. Sparrow.

The writer is not aware that any Court has laid down the principle, which this recommendation of the Committee involves, that the public should guarantee a company against loss from operation, and he doubts if such a principle will ever be adopted, but he feels sure that no Court will ever lay down the principle that such losses constitute a legal investment in the business of the company on which the public is obligated to pay a return. Although refusing to admit the principle that the public is obligated to refund the losses of a company made in the first years of its life, the writer believes that where a company, in the prosecution of legitimate business, has contracted losses in the starting of its business, or where the returns have been insufficient to pay a fair return on the investment, and the company can conclusively prove that it has not recouped such losses in later years, it would be fair and just in fixing the rates to make the return such that the company would be recompensed for such losses. In other words, the writer cannot conceive of such losses as an investment in a plant or business, but thinks that it is a condition which may justify a higher rate than would have been the case had no such losses been incurred, until such losses have been amortized.

The writer considers that if the principle were adopted that the deficiency, representing the difference between the amount actually earned and the amount which the corporation is entitled to receive, under the basic principle that it is entitled to a fair return on the fair value of its property, is to be capitalized and treated as an investment on which the public shall pay a fair return, then, as a corollary, the surplus, representing the difference between the amount actually earned and the amount which the corporation is entitled to receive under the same basic principle already outlined, should not only be returned to the public, but the corporation should continue to pay the public a fair return on this amount. The writer believes that on past records the public could well afford to adopt this principle.

Land Values.—In view of the physical valuation of the railways in the United States, which is about to be made by the Interstate Commerce Commission, this question looms up very large, and to

Mr. Sparrow. the writer's mind overshadows all other questions in the valuation of public utility corporations. The tremendous importance of the question can be better appreciated by a reference to one or two recent cases involving this question. In the Minnesota Rate Case the total original cost to the Northern Pacific Railway Company of the terminal properties in Minnesota was $4 527 229. The Master in the United States Circuit Court allowed a value of $17 315 869. The total original cost of the entire system was found to be a little more than $312 000 000, but the cost allowed by the Master on the reproduction basis was more than $452 000 000, the difference, $140 000 000, being principally the increased value of the land.

In the Western Advance Rate Case, the Burlington claimed a return on a present value of $530 000 000. Commissioner Lane found that the amount of the original investment was only $258 000 000, and that about $150 000 000 of the Burlington's claim represented increase in the land values.

In the appraisal of the Northern Pacific Railway by the Washington Railroad Commission, H. P. Gillette, M. Am. Soc. C. E., Consulting Engineer to the Commission, placed the original cost plus improvements at $75 457 893, and the cost of reproduction new at $103 613 442. The difference is mainly due to the increase in the value of land.

These cases are only typical of many others, and they illustrate powerfully the magnitude of the question under discussion.

The claim on which the right to capitalize the increased value of the land is based, is that the practice in recent years has been in favor of the reproduction cost less depreciation, and that the fair value on which the return is to be made is the present value rather than the original cost, and that, on this basis, land must be treated in the same manner as the other property of the corporation.

If we study the history of valuation methods and Court decisions, it is easy to see how the reproduction cost or present-value theory came to birth. In the first cases which came before the Courts, the utility corporations claimed a return on the amount of stocks and bonds outstanding, which, it was argued, represented the original cost plus improvements, or the investment in the property. The Courts appear to have seen very clearly that to allow a return on such a basis would be unfair, for the very reason that such stocks and bonds in many cases included water, and did not represent the investment in the property. At this time there did not appear to be any difference of opinion in the minds of either the corporations or the Courts that the return should be based on the investment, and the only reason for the Courts' refusing to allow a return on the outstanding stocks and bonds was that they did not represent the *bona fide* investment. In searching for a way out of the difficulty, the reproduction-cost and present-value methods were devised. At the time it does not appear to have been

foreseen by the Courts that in avoiding the frying-pan they may have fallen into the fire, and that much greater injustice may be done the public in the application of these theories to the unearned increment of land. Mr. Sparrow.

Land differs very materially from the other property of a utility corporation for the reason that it seldom if ever depreciates. There may be times when the value of land does not increase, but, if taken over any considerable length of time, it will in nearly all cases increase. In the case of a utility corporation, it will invariably increase, and that enormously. Here, therefore, the application of the reproduction theory is very one-sided, and is as correspondingly unjust to the public as it is advantageous to the company.

It is recognized and accepted beyond further argument that the consumer must pay such a return to the company as will cover the amount of the depreciation of the company's property. This being so, and the consumer being debited with the amount of depreciation, he must, as a matter of substantial justice, be credited with the amount of the appreciation.

This contention is supported by the general rule laid down by Judge Hough of the United States Circuit Court in the case of the Consolidated Gas Co. *v.* City of New York. Judge Hough says in part:

"Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the case at bar."

Commissioner Maltbie has attempted to consider appreciation in value as income, canceling it against depreciation in other property, but the writer believes that the same result would be arrived at more logically and with greater simplicity by appraising the land at its original cost. The railroad is a public servant. It has been thus defined by the Supreme Court, but the writer prefers to consider it as the agent of the public.

In the case of Smyth *v.* Ames, Mr. Justice Harlan says:

"A railroad is a public highway, and none the less so because constructed and maintained through the agency of a corporation deriving its existence and powers from the state. Such a corporation was created for public purposes. It performs a function of the state. Its authority to exercise the right of eminent domain and to charge tolls was given primarily for the benefit of the public."

To the writer, the answer to the whole question is contained in this weighty and forceful declaration. If a railroad company or other public utility is the agent of the State, performing a function of the State, and in that guise is vested with a governmental authority to acquire land for the purpose of serving the public, can this fundamental relationship, to which the company owes its very existence, now

Mr. Sparrow. be thrown aside, and the agent demand from the principal the increased value on the land it was allowed to acquire for the purpose of serving and benefiting the principal.

The land was never granted to the railroad company for real estate speculation. It was granted for one purpose only—railroad purposes, and for the benefit of the public.

If the rate is insufficient for the service performed, and does not yield a fair return on the money devoted to that service, then it is an argument for an increase in that rate, and, in justice to the company and in the interest of the public, the rate should be increased; but, if the rate is sufficient for the service performed and does yield a fair return on the money invested, then can it be considered as an argument for an increase of that rate that the real estate has increased in value and a further toll be imposed on the public to whose service this real estate was dedicated?

The writer is strongly opposed to Government ownership of public utilities, but he is a firm believer in Government ownership of land, and that the Government, State or Federal, as the case requires, should acquire all land necessary to the construction and operation of a public utility and lease such land at a fair return on the cost of acquiring it to the company, such rental being charged to the operating expenses of the company and paid by the consumer. If such a principle had been adopted from the start, the question of the unearned increment could never have arisen, as the increase in value of the land would have been credited to the account to which it rightfully belongs, that of the public.

Mr. Wilgus. WILLIAM J. WILGUS, M. AM. SOC. C. E. (by letter).—Although the writer has the gratification of knowing that the views expressed in his recent* paper on the physical valuation of railroads are substantially in accord with those contained in this very complete and admirable report, he finds himself unable to agree to the proposition that, for rate-making purposes, depreciation should be deducted from the cost of reproduction new.

The Committee considers that depreciation should be divided into two classes, the first consisting of low-cost and shorter-lived items of property to be repaired or replaced through charges to current expenses; and the second consisting of high-cost and longer-lived items of property, to be renewed through charges to capital.

Therefore, apparently, it is the intention of the Committee that the first class is to be treated as an operating expense and the second class as a restoration of capital to the investor.

If this is a correct understanding of the Committee's position, the sum that it proposes shall be taken as the basis for rate-regu-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

lation is not "cost of reproduction new less depreciation," but rather "cost of reproduction new less the portion of depreciation that applies to high-cost and long-lived items." In this event, the portion of depreciation that is to be taken care of through current expenses is not to be deducted from cost of reproduction new. Mr. Wilgus.

Even though common practice and the rules of regulatory commissions should permit the charging of replacements of a large nature to capital account, which they now expressly prohibit, it is difficult to see how in practical every-day railroad accounting and maintenance it will be possible to draw a well-defined line between the two classes of depreciation.

The Committee says:

"As a general statement, it may be said that the basis of expectation of life should be used for all important items of property having a life of more than, say, five years, except where it may involve unwarranted expense for accounting or be impracticable to separate what would properly be, under the Equal-Annual-Payment Method, charges to current expenses and to capital."

The writer believes that on railroads with their vast multitude of parts of varying and uncertain lives, the exceptions mentioned would prove the rule and the plan would fall. Rails and ties on sharp curves or of inferior material or subjected to heavy traffic, may have a very short life, though at other places less exacting conditions will make for a greatly lengthened life. An increase in the weight of motive power suddenly may cause bridges to become obsolete, whereas a change from steam to electric locomotives, with their lessened weights, may as sharply reverse the situation. Buildings and wharves under certain circumstances or policies may be continued in use indefinitely, through repairs and remodeling; and under other conditions may be replaced with new structures; or they may be abandoned. The instances may be multiplied *ad infinitum* to show that the drawing of a line between the two classes of depreciation would prove to be impracticable.

With the admission by the Committee that, because of certain difficulties, only a portion of depreciation may with propriety be deducted from cost of reproduction new, the grounds for asserting as a principle or fact that a depreciation allowance is a return of a part of the investment would seem to lose force.

The crux of the question appears to lie in the requirement of the Interstate Commerce Commission and the various State regulatory bodies that upkeep shall be charged to expenses and not to capital. As long as this situation continues, depreciation plainly is a liability of the owner to be replaced by him out of the allowance made for that purpose in the rate, or in case of misuse of such

Mr. Wilgus. allowance, to be made good by him out of the part of the rate assigned to him for profit.

In this connection, sight should not be lost of the impossibility of maintaining a railroad in a new condition. A well-maintained property in time reaches an average condition, say, 85% or 90% of new, which remains practically stationary. In such cases an appraisal with depreciation deducted will show a loss of value, though, as a matter of fact, the real worth will be greater. For instance, no one will deny that the Pennsylvania Railroad between Jersey City and Philadelphia is more valuable for the purpose for which it is used, than a brand-new unseasoned parallel line having the same physical characteristics. Depreciation in cases like this is merely a measure of the cost of securing a seasoned railroad and may be considered as belonging in the same category as other legitimate overhead expenses.

The importance of this depreciation question is so great that it is to be hoped that the Society will not lightly lend its sanction to any view that will not successfully bear a critical analysis.

The Committee considers that adaptation and solidification should have no place in the valuation, unless they have involved the expenditure of money. As no such rule is proposed in the cases of donations of right-of-way and other property, it hardly seems logical to exclude the enhanced value that comes with time to the seasoned railroad, nor does this course seem equitable when the stand is taken that depreciation due to the action of the elements and natural causes shall be deducted.

Mr. Gibson. J. E. Gibson, M. Am. Soc. C. E. (by letter).—The Society is fortunate in obtaining this valuable report, and the gentlemen composing this Committee are to be congratulated on their thorough and painstaking work. It is to be hoped, however, that the Society will give careful consideration to this report and before presenting it to the public as its report, will consider every phase of the matter, so that all parties in the proposed valuation, if conducted under the Society's rules, will have justice and equity. Undoubtedly, any rules that the Society may formulate for procedure in valuation cases will carry great weight with the public and the Courts. The report, the writer believes, does not insure complete justice and equity to the privately owned utilities.

The report is very comprehensive, but there are several points on which the writer cannot agree with its conclusions. It has running through it that deference and awe that all have for the majesty of the Court. The writer does not in all cases share this deference, as he recognizes the Court at best as only human, and as likely to err as others. Necessarily, the Court lives in the past, and its actions are controlled by precedent, whereas the engineer must of necessity

be a pioneer and establish his own precedents and landmarks. This being the case, the Courts have erred unintentionally and honestly in many of their decisions in the valuation of public utilities. To correct these conditions and to place before Courts and commissions proper rules for procedure is one of the functions of the members of this Society. Men trained in engineering subjects and having wide engineering experience in construction, in conjunction with men trained in financing and operating public utilities are certainly more competent to pass on the valuation of these utilities than those trained in the law, such as the Courts. Mr. Gibson.

Purposes of Valuation.—The Committee states in one of the opening paragraphs of the report as follows:

"It is sometimes held that valuations for all of these purposes should give identical results. This view cannot prevail if the laws of the different States and certain decisions of the Courts are complied with."

It seems to be a common belief that the value of a property depends on the purposes for which the valuation is made, but the longer this is considered the more one is forced to the conclusion that the true value of a property is independent of the purposes for which the valuation is made. If this premise be correct and the decisions of the Courts and the laws of the various States are antagonistic to these premises, then it is our duty as a body to take all reasonable and just measures to modify and correct the laws and decisions of the Courts. The value of a property for the purposes of sale should be the same as that for the purpose of rate-making or taxation; for, should the purpose be that of sale, and a sale be consummated on the valuation made, then the new purchaser should be entitled to receive a proper return on the purchase or valuation price, and this return, in justice to all parties, should be neither greater nor less than that on the valuation of the property before sale for rate-making purposes.

It is realized that a property may have a sentimental value, but no purchaser is willing to pay a greater price because of such sentimental value. On the other hand, under certain conditions, a purchaser will pay more than the apparent real value of a property owing to a latent or potential value, which, by his business ability or connections, he can transform to an active value, but, in so doing, he converts the latent value to the uses of the public, and is therefore entitled to a proper return on this increased value.

The Committee touches on what would be termed the negative phase of this latent value where it speaks of a property, which, under certain abnormal conditions, may never be able to earn a fair return on the investment, and says "such a property may be classed as a losing venture". In valuing a property, should it have a latent value

Mr. Gibson. due to strategic position or other natural facilities, this value should be added to the physical value, the same as an appreciation in the value of its real estate, and, on the other hand, by unfortunate location, wasteful construction, or otherwise, should the property be unduly expensive, then this inherent and excessive cost should be deducted from the valuation, the same as depreciation on the physical plant would be treated.

Methods of Determining Physical Value.—A board of public utility commissioners or engineers should strive to place themselves in the position of the original builders at the time the plant was conceived. It has been the writer's observation that with but few exceptions all engineers and contractors are optimists, and are always willing to undertake to do for less money, and in a shorter time, more work than they can actually perform. Therefore, in valuing an existing plant, and notwithstanding that they have a life-size plan of the work to be valued, the appraisers are more than likely to lose sight altogether of the contingent expenses and items that go to make up a completed whole—such as the difficulties of labor, non-delivery of machinery, conditions of weather, difficulties of financing, and the other multitudinous delays that occur in the construction of extensive works. They should always be very liberal in their estimates and criticisms of the property, and give the benefit of every doubt to the owner of the property. To this end they should use great care and diligence in reaching their final conclusions, and, for this purpose, should always seek the advice and counsel of men who have had experience and training in the actual construction of such works. It is obviously unfair to ask a railroad engineer to pass on the cost of a water-works property. He (the railroad engineer) would unintentionally, but honestly, undervalue an embankment for reservoir purposes, as the methods of construction are entirely different, and, in the same manner, it would be unfair to the railroad property to ask a water-works or hydraulic engineer to value a railroad property. It seems, however, to be the practice of most of the present public utility commissioners to employ one chief engineer and expect him to pass on all classes of utilities. This procedure, in a great many cases, works an injustice on the majority of the utilities in the State. Again, on account of the niggardly policy of some of our legislatures in the payment of its officers, the commissioners are very often compelled to accept the services of men who have not had the necessary experience in the actual construction and operation of public utilities, and therefore do not realize the cost of performing satisfactory service.

Unused Property.—This is a rather difficult subject, but if the property that is not in use has been purchased with a view of future extensions and use, and this future use or extension will be required

within a reasonable time, the company is entitled to a return on this property; and, in line with this reasoning, a public utility is entitled to a return on a plant which is built larger than the immediate needs require. Of course, the appraisers should use their discretion as to whether this unused property or over-built plant is beyond the reasonable requirements and needs of the future, but, even in this case, the utility should be given the benefit of the reasonable doubt. A water-works utility company, for instance, is certainly entitled to construct a plant of sufficient size to meet the public need for a period of at least 10 years in advance of the immediate requirements, and any outlays of money looking toward the extension of its piping system, pumping facilities, or water supply, with this object in view, are certainly within legitimate business requirements, and one which the public should demand of the utility. Mr. Gibson.

Present or Original Conditions.—The board of appraisers, in considering the present conditions of the plant, should also consider the conditions existing previous to the commencement of construction; otherwise, the utility might be done a great injustice.

The writer does not agree with the Committee regarding the increased value of the plant due to public improvements, etc. Under existing laws, the holder of real estate or other property is entitled to the unearned increment of value from his property, and the same should hold for a public utility. Because the utility has been given something outright does not relieve the public from paying a fair return on this property, provided it receives the benefit of the use of the property: Because A buys an auto truck for $10 000, and then turns it over to B for nothing does not impose on B the operation of this truck for the benefit of the public without a return. This is a parallel case to that of a public utility which has had service connections and rights of way donated to it, or that of the public constructing highway improvements in its territory. The writer, however, believes that the service connections of the patrons should be put in at the expense of the utility, and the utility should be permitted to carry this charge as a capital charge, the same as for any other portion of the property. It is unfair to exclude the cost of these improvements in the valuation of the plant for rate-making purposes, for, if the plant were being valued for sale, these items would enter into the valuation, and, therefore, should the sale be consummated, an injustice would be done the new purchaser, as he would not be allowed a return on these portions of the purchase valuation, which would defeat the purposes of the valuation for rate-making, namely, equity to the owner and equity to the public.

Reproduction Method of Valuation.—The writer can see no objection to the reproduction method of valuation for existing public utilities, and, for properties that have been constructed previous to

Mr. Gibson. the last decade, it is probably the only method that can be adopted satisfactorily. In the hands of engineers of experience and judicial temperament it will give a valuation equitable to all parties concerned.

The writer cannot fully agree with the report as to the propriety of deducting the depreciation on the existing plant. If the plant has earned a competent return sufficient to pay interest charge, operating expenses, and depreciation, then it is entirely proper to reduce the reproduction value of the plant by the amount of the depreciation; but, should the plant not have earned a competent return, but have gone along from year to year, paying only interest and operating expenses, it has done so at the expense of its vitality and due to its substantial construction. The public has received the benefit in low rates due to the plant's vitality, and therefore it is only just to the owner that he should be allowed to receive an additional return, which, on an amortization basis, would cover the accrued depreciation, at the time of valuation, at the expiration of the normal life of the plant. It is realized that this is not placing the burden on those who should have borne it, but it is the best that can be done; otherwise, the owner is compelled to put new capital into the plant to meet the demands of the public, or to suffer a total loss of that portion of his capital, or be willing to accept a return that is not a reasonable one. We are justified in placing this additional burden on the present and future generations by common consent; witness the issuing of long-term bonds for public improvements, etc.

Unit Prices.—The use of the average prices of material for periods of 5 or 10 years, ordinarily speaking, is fair, but occasionally there are times when this method would do an injustice to one of the parties. For instance, during the past few years, the price of cast-iron pipe has been as high as $36 per ton, and it is now approximately $24 per ton, a reduction of 25 per cent. A water-works property properly designed and constructed in 1906 and 1907, when pipe was selling at from $30 to $36 per ton, is just as capable of performing service as if constructed to-day; therefore, to value it on the price of pipe since 1907 would be an injustice to the water company, and then again, a plant constructed in 1898 when cast-iron pipe was down to $18 per ton, valued in 1906 at $30 per ton would be unjust to the public, so it is again seen that the judgment and discretion of the appraising board must be used if equity between all parties is to be obtained.

The writer believes that the prevailing rates for labor, during the previous 12 months, in the vicinity of the works that are being valued, should be used, as the rate of wages for labor is constantly increasing and will probably never be less than it is at present.

Overhead Charges.—The writer endorses fully the report with reference to the items of overhead charges, as with few exceptions, they are probably universally under-estimated. Preliminary promotion and engineering expenses are in a great many instances paid out of other resources than those of the company, and, due to the optimism of the engineers, the preliminary and organization expenses are grossly under-estimated. The writer ventures to state that very few engineers realize the actual cost of reproducing their plans and blue prints covering any extensive work. Their cost records in this department of their work are with few exceptions neglected. Mr. Gibson.

Development Expenses.—In this the Committee gives a new term for an old one without aiding or elucidating the difficulties that have been met with in the use of the old term. The discussion of "going value", as applied to a utility, has probably covered more space in the proceedings of the various technical societies than any other single subject in recent years. Some claim that it is nothing but a guess; others that it is another way of expressing the deficits of the company that have accrued from the time of its completion to the date of the valuation. Charles Gobrecht Darrach, M. Am. Soc. C. E., argues that "going value" is a misnomer, and that the proper term should be adolescence, comparing it to the growth and development of a child from birth to manhood. Undoubtedly, an operating plant has a greater value than a non-operating plant, and whatever this value may be called, the utility is entitled to earn a return on it, as it represents an expenditure of capital or its equivalent. Whatever the term used for this value, the writer would define it as that value of an existing operating plant having consumers and revenue over and above the value of a similar plant in the same locality having the ability to serve but no consumers or revenue. This value is independent of whether or not the utility is earning a satisfactory return. It is a value, however, that is susceptible of quite a bit of variation, but if approached with an open and judicial mind, by appraisers of skill and experience in construction and operation, it will give equitable results to all parties.

Depreciation.—The writer regrets that the Committee finds it necessary to offer a new name for the depreciation allowance. It offers the new term, "Equal-Annual-Payment Method", which is just as difficult to explain as the sinking-fund method, and, if not read carefully, gives one the impression that it proposes to repay to the investor a portion of his capital annually and thereby reduce his interest return. The equal-annual-payment method is substantially the same as the sinking-fund method, except that the latter requires that the annual amount of depreciation be used in the business, and therefore, the writer would much prefer the term "Uniform-Investment-Charge

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Method", used by Mr. Robert H. Whitten.* By this method the amount of the depreciation is put back into the works and bears the same return as the other portions of the capital, and the advantages are that it insures to the investor a constant one hundred cents on the dollar value in the property and at the same time permits the public to obtain a minimum charge for service rendered, as the amount of the depreciation of the original cost is reinvested in the property and is made to earn the same interest charge as the original capital.

The amount of this depreciation, however determined, should not be deducted from the value of the plant when fixing the rate of return to be allowed the utility, but only considered in determining the additional return to be allowed to cover the wear and tear, obsolescence, and inadequacy. It is obvious that, should the utilities be allowed only a return on the depreciated value of their properties, they could not meet their interest charges after the first year.

The writer's objection to any of these methods is the multiplication of terms and the difficulty in explaining to the investing public, and possibly the banker, the method proposed to be followed in allowing for depreciation, whereas the sinking-fund method is generally understood. The results obtained by either method are identical, provided the depreciation charge is put into the business and not returned to the original investor, or invested in outside securities.

The term "Uniform-Investment-Charge Method" by its name conveys the idea that the investment is maintained at a uniform value and that the charge to the rate-payers is uniform.

Appreciation.—The writer agrees with the Committee with reference to appreciation, notwithstanding the fact that quite a few recent Court decisions have viewed it differently. Certainly, if a utility asks for a return to meet depreciation, it is only fair to the public to grant them a return for any appreciation in the value of its works due to the growth of the community, etc.

Land Valuing.—Any engineer of experience in the promotion and construction of public utilities is fully aware of the appreciation in land value due to the knowledge that it is to be used by a public corporation. The owner of land, with few exceptions, on learning that his property is required by a utility or corporation, at once places an inflated price on it. Further, the prospective utility can afford to pay almost any price rather than proceed by law to condemn under the provisions of eminent domain, as the general public views all utilities as legitimate spoils. Of course, an owner of property is entitled to a reasonable valuation for the property taken, together with the consequential and severance damages, but, in the majority of cases, boards of jurors have taken a too liberal view and awarded inordinate damages. Therefore, a board appraising public utilities

* Ably set forth in *Engineering News* May 8th, 1913.

should be equally liberal in valuing the property of the utility, and for this purpose should consult with and obtain the views of real estate men having to do with the purchase of properties for the class of utilities being appraised. Mr. Gibson.

Rates Based on Physical Value Only.—The writer heartily endorses the opinion of the Committee that the rates should be based on other items than the bare physical value of the plant, and, in this line, thinks that it should certainly cover the rates on such intangible values as strategic location, efficiency, etc. He, however, does not believe that, because a utility has earned exceptional rates, due to its non-regulation by utility committees, it should continue to do so in the future, although justice to the innocent investors would require that their rates be reduced, not at one sweep, but rather on a sliding scale, so as to allow a sufficient time for readjustment.

Continuous Regulation by Commissions.—The writer agrees with the Committee that the regulation should be continuous and not spasmodic, but it absolutely fails to touch on the point of how often the regulation and readjustment of rates should be undertaken. In justice to the public and the investor, the rates should be adjusted to cover as long an interval as possible, and the writer had hoped that the Committee would make some recommendation as to the desirable interval between adjustments, as at present the companies can be made to defend themselves yearly before the commission. This means unnecessary expense and harassment to the company, deters public investors, and accomplishes no beneficial results. The Committee has not covered the question of the treatment of the expenses of financing, possibly thinking it was not necessary to do so. Neither does it discuss the question of intangible assets.

It has been generally decided that discount on bonds should be amortized. Therefore, in figuring the returns on the property, the interest rate should be increased to permit of this amortization fund. There is certain intangible property, which should also be amortized—such items as legal expenses, preliminary engineering, and promotion—for all these expenses ceased to exist at the expiration of the physical life of the plant, and a sufficient return should be earned so as to enable the utility to write these charges off its books at the expiration of a certain interval of time.

The Committee does not touch on the phase of the publicly owned utility, but seems to confine itself entirely to that of the privately owned. With few exceptions, the present laws of the various States have exempted the publicly owned utilities from regulation. This does a great injustice to the private utility. The latter is, to say the least, the agent of the State as much as the public utility, and therefore, though a publicly owned utility should not be permitted to earn a profit, it should be made to earn rates sufficient to cover

Mr. Gibson. operating expenses, interest, and depreciation, and not be allowed to make up its deficits out of the general tax fund. If this had been made compulsory in the past, the public would more readily recognize the benefits that have been conferred on it by the privately owned utility.

Public utilities, by their immobility and the nature of their business, must of necessity be monopolies, and therefore, to permit duplication of utilities is the grossest kind of economical waste, and should not be permitted under any circumstances.

In conclusion, the writer had hoped that the Committee would adopt a definite nomenclature for the various terms used in valuations, giving clear and concise definitions of the terms used. This is one of the first problems that should be undertaken in formulating and standardizing a method of procedure on so great a subject as this.

Mr. Vensano. H. C. VENSANO, ASSOC. M. AM. SOC. C. E. (by letter).—This report has been read with great interest. It should be of great value to all interested, and seems to go very thoroughly and clearly into all parts of the question. There is still, however, in the writer's mind, some doubt as to the proper method of handling depreciation.

It would seem to the writer, after going over this matter, that all methods thus far suggested become practically one after a number of years of application to a particular property. The only difference seems to be in the amount finally arrived at as the depreciation increment.

In substantiation of this, two tables are submitted: Table 1 is drawn up to correspond to the table on page 34 of the report, the only difference being that Table 1 has been made up on the basis of a property having an assumed life of 5 years. This is given for purposes of comparison with Table 2. A property having a life of 5 years has been used for simplicity and in order to shorten the calculations. Table 2 considers a complex property in which the annual depreciation increment can be at once reinvested. The left half of this table shows the remaining value of the property from year to year on which it is allowed to earn an investment return. The right half shows the depreciation increments reinvested each year in accordance with the Committee's suggestion that such "depreciation allowance" should be "treated as a part of capital and used for any purposes for which the corporation is authorized to use capital". Each column in turn then shows the depreciation allowances on the reinvestment of the preceding year's allowance. In the last column of the left half of Table 2 it is shown that the total value of the property on which interest is allowed to be earned has been maintained at $100, or its full original value, so that $5 is constantly being earned as an investment return.

In the final column on the right of Table 2 is shown the total depreciation allowance for each year. Mr. Vensano.

TABLE 1.—ILLUSTRATION OF EQUAL-ANNUAL-PAYMENT METHOD OF COMPUTING DEPRECIATION.

(1) Age, in years.	(2) Value at end of year.	(3) Depreciation during year.	(4) Return on Remaining Value of Property at: 5%	(5) Return on Remaining Value of Property at: 7%	(6) Combined Depreciation and Return on Investment at: 5%	(7) Combined Depreciation and Return on Investment at: 7%
0	$100.00					
		18.0975	$5.0000	$7.0000	$23.0975	$25.0975
1	81.9025					
		19.0023	4.0952	5.7332	23.0975	24.7355
2	62.9002					
		19.9525	3.1450	4.4030	23.0975	24.3555
3	42.9477					
		20.9501	2.1474	3.0063	23.0975	23.9564
4	21.9976					
		21.9976	1.0999	1.5398	23.0975	23.5374
5	0.00					
		$100.0000				

Assumptions: Property having 5-year life valued when new at $100. Computations of depreciation allowances based on 5% interest compounded annually. Annual return on capital invested at 5 and 7 per cent.

Particular attention is called to this right hand column, from which it can be seen that at about the fifteenth year's life of a property, the component parts of which have an average 5-year life, the annual depreciation allowance, under the equal-annual-payment method, so-called, has become constant and practically remains so during succeeding years. In other words, both investment return and depreciation allowance have become constant and we have arrived automatically at a straight-line or railroad method. The amount arrived at, of course, is different, but the method has automatically become the same. It is to be noted, also, that this is absolutely the same result that would be arrived at by following out the sinking-fund method, where the sinking fund has been reinvested in the property, so that equal rates of return have been obtained on the sinking fund and the original property.

In Table 2 the same rate of return, namely, 5%, has been allowed on both the original investment and in figuring the sinking-fund increment, and the writer does not agree with the Committee in advising that these rates be different, because it is apparent that, by reinvesting a sinking-fund increment in the property itself, the same rate of return as on the original property can be obtained.

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Return on value of investment at 5%.	Total value of investment.	Value of Capital Reinvested.														
		16th year.	15th year.	14th year.	13th year.	12th year.	11th year.	10th year.	9th year.	8th year.	7th year.	6th year.	5th year.	4th year.	3d year.	2d year.
$5	$100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	22.27
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	27.42	18.24
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	33.75	22.48	14.00
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	41.56	27.63	17.26	9.56
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	28.06	34.05	21.21	11.78	4.90
5	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	30.35	22.97	26.15	14.48	6.03	.....
5	100	.....	.....	.....	.....	.....	.....	.....	.....	32.21	24.87	17.64	17.85	7.42	.....	.....
5	100	.....	.....	.....	.....	.....	.....	.....	33.33	26.40	19.10	12.04	9.14	.....	.....	.....
5	100	.....	.....	.....	.....	.....	.....	33.22	27.29	20.28	13.04	6.17	.....	.....	.....	.....
5	100	.....	.....	.....	.....	.....	31.31	27.22	20.96	13.85	6.68	.....	.....	.....	.....	.....
5	100	.....	.....	.....	.....	32.07	25.65	20.90	14.31	7.10	.....	.....	.....	.....	.....	.....
5	100	.....	.....	.....	32.47	26.27	19.70	14.27	7.33	.....	.....	.....	.....	.....	.....	.....
5	100	.....	.....	32.51	26.58	20.17	13.45	7.31	.....	.....	.....	.....	.....	.....	.....	.....
5	100	.....	32.32	26.62	20.41	13.77	6.89	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	100	32.11	26.47	20.44	13.93	7.05	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Assumptions:

Property, 5-year life.
Value of property when new = $100.
Depreciation allowance based upon 5 per cent.
Interest compounded annually.
Annual return on capital invested, 5 per cent.

Under Government regulation and fair adjustment of rates, such reinvestment should presumably be entirely safe, and should, therefore, without question, be entirely permissible.

In explanation of the construction of Table 2, it might be well to state that Column 22 is the depreciation return from year to year on the original investment. Column 23 assumes the reinvestment of the $18.10, which was the depreciation return for the first year, and shows the annual depreciation return on this $18.10. Column 24 then, similarly, shows the depreciation allowance on $19 (being the depreciation allowance on the original investment for the second year), plus $3.27 (being the depreciation allowance on the reinvestment of the first year, $18.10). Each column in turn then represents depreciation allowances on a reinvestment of all depreciation allowance accrued during the preceding year. The left side of the table, similarly, shows the remaining values for each year's reinvestment. These computations have been made with the slide-rule, and the sum of the items may not quite check with the required totals.

The writer feels that some confusion has obtained in the past in the handling of suggested methods of depreciation allowance because of

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1st year.	Value of original property.	Age, in years.	Depreciation of original property during year.	Depreciation Allowance on Preceding Years Reinvested.																Total yearly depreciation.
				1st year.	2d year.	3d year.	4th year.	5th year.	6th year.	7th year.	8th year.	9th year.	10th year.	11th year.	12th year.	13th year.	14th year.	15th year.	16th year.	
.....	100.00	0	.....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	..	.....
18.10	81.90	1	18.10	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	..	18.10
14.83	62.90	2	19.00	3.27	....	....	....	....	....	....	....	....	....	....	....	....	....	....	..	22.27
11.38	42.95	3	19.95	3.44	4.03	....	....	....	....	....	....	....	....	....	....	....	....	....	..	27.42
7.77	22.00	4	20.95	3.61	4.23	4.96	....	....	....	....	....	....	....	....	....	....	....	....	..	33.75
3.98	0	5	22.00	3.79	4.44	5.22	6.11	....	....	....	....	....	....	....	....	....	....	....	..	41.56
.....	......	6	.....	3.98	4.66	5.48	6.42	7.52	....	....	....	....	....	....	....	....	....	....	..	28.06
.....	......	7	.....	....	4.90	5.75	6.73	7.90	5.07	....	....	....	....	....	....	....	....	....	..	30.35
.....	......	8	.....	....	....	6.03	7.06	8.30	5.33	5.49	....	....	....	....	....	....	....	....	..	32.21
.....	......	9	.....	....	....	....	7.42	8.71	5.60	5.77	5.83	....	....	....	....	....	....	....	..	33.23
.....	......	10	.....	....	....	....	....	9.14	5.87	6.06	6.12	6.03	....	....	....	....	....	....	..	33.22
.....	......	11	.....	....	....	....	....	....	6.17	6.36	6.43	6.33	6.02	....	....	....	....	....	..	31.31
.....	......	12	.....	....	....	....	....	....	....	6.68	6.75	6.65	6.32	5.67	....	....	....	....	..	32.07
.....	......	13	.....	....	....	....	....	....	....	....	7.10	6.98	6.63	5.95	5.81	....	....	....	..	32.47
.....	......	14	.....	....	....	....	....	....	....	....	....	7.33	6.96	6.25	6.10	5.87	....	....	..	32.51
.....	......	15	.....	....	....	....	....	....	....	....	....	....	7.31	6.56	6.40	6.17	5.88	....	..	32.32
.....	......	16	.....	....	....	....	....	....	....	....	....	....	....	6.89	6.72	6.48	6.18	5.85	..	32.12

the apparent endeavor to fit them to the "necessary monetary reinvestment for depreciation" rather than to the "actual depreciation" itself. He believes that there should be a clear distinction between these two, depreciation being thought of rather as the actual inherent curtailment of life of the various parts of the property, whereas the monetary replacement requirements may be something very different.

In general, in a large and complex property, the true depreciation is very nearly a straight line, whereas the monetary requirements for replacement of its component parts may vary greatly. In general, for the first few years of life, such requirements would seem to increase from little or nothing upward, and gradually approach an average, although annually fluctuating above and below such average.

For instance, assuming a property in which the longest-lived single item would have a life of 10 years and would have other items of lives of 1, 2, 3 years, etc., up to 10 years. At the end of the first year the depreciation on the item of 1 year's life would be 100%; on the item of 2 years' life, 50%; on the item of 3 years' life, 33⅓%, etc. At the end of this year the item of 1 year's life, of course, would have to be replaced, and at the end of the second year we would again have 100% depreciation for the item of 1 year's life, 50% for the item of 2 years' life, 33⅓% for the item of 3 years' life, etc. At the end of this year the items of 1 and 2 years' life would have to be replaced, and would be so replaced. Thus, at the end of the third year we

Mr. Vensano. would again have the actual depreciation for each class of items as before. Now, note the variation in replacement requirements. At the end of the first year we must replace all the items of 1 year's life; at the end of the second year all the items of 1 and 2 years' life; at the end of the third year all the items of 1 and 3 years' life; at the end of the fourth year all the items of 1, 2 and 4 years' life; at the end of the seventh year all the items of 1 and 7 years' life.

For properties having parts of extreme lives, say 100 years, we might have replacement in the eightieth year, for instance, of properties of 1, 2, 4, 5, 8, 10, 16, 20, 40 and 80 years' life; whereas, in the seventy-first year, for instance, we would only have to replace the items of 1 and 71 years' life, showing a great range in the replacement requirements.

It might seem, therefore, that, considering the "actual depreciation", the straight-line method might work out most simply and still be entirely logical, the only objection being that, for the first few years of the property's life, a too high depreciation allowance would have been made to cover "monetary replacement requirements", but, as the property ages, the results obtained would be very nearly correct, as shown by Table 2.

Although making this statement, the writer still believes that the sinking-fund method is the preferable one, and should be used. Of course, he realizes that, because of Court rulings, rates must be fixed on depreciated property values, and the use of this method becomes somewhat difficult, but he thinks that by proper presentation of cases in future such decisions might be reversed and it might be allowed (where properties have been properly handled) to base figures for rate-making purposes on the undepreciated value of the original properties, and thus make this method usable.

The Committee's suggested method gives the same results, as previously stated, as the sinking-fund method, and the Committee has no doubt met the Court requirements, but, as just shown, it would seem that the same result could be arrived at by the straight-line method with less complications in the figuring of the depreciation allowances, and with a very material simplification of corporation accounting, which it would seem would become very complicated under the suggested method.

Mr. Crehore. WILLIAM W. CREHORE, M. AM. SOC. C. E. (by letter).—The writer is in accord with the Committee's views on all the important features of this report; the only suggestions he has to offer relate to matters of, perhaps, minor significance, but which, if modified, might be a greater force in convincing those who are opposed to the principles and methods outlined by the Committee.

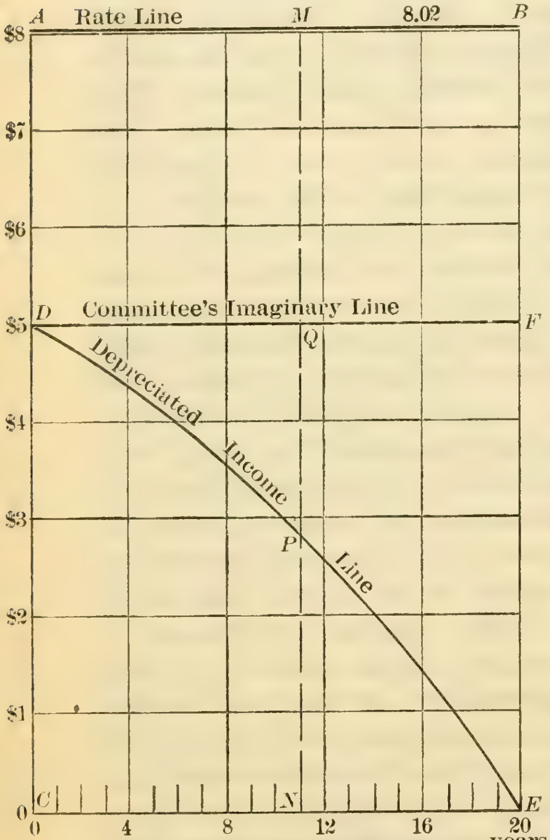
Choice of Name.—As was well known, the real determination of the rate involves the whole theory by which a depreciation allowance is to be made. The Committee advocates what it calls the Equal-Annual-Payment Method, asserting that by its use the same results will be obtained as by the so-called Sinking-Fund Method, and then proceeds to show that, if depreciation takes place at a 5% compound-interest rate, the sum of the allowed depreciation during any one year and a 5% return on the remaining value of the property will provide a constant rate throughout the whole life of the property. The writer agrees with the Committee's theory of depreciation allowance, but thinks the adopted name, Equal-Annual-Payment Method, is a misnomer. Very great confusion has arisen in the past, as the Committee points out, from a misunderstanding of the difference between the Sinking-Fund Method, as such, and methods which make use of the sinking-fund curve. There seems to be no reason for this, except that the Committee's method and the Sinking-Fund Method both depend on a compound-interest rate of depreciation. Surely, many other things may depend on a compound-interest rate of change besides sinking funds, and just because a sinking fund does thus depend for its values from time to time is no good reason for not using a compound-interest rate when desirable, without the accompanying thought of a sinking fund. Mr. Crehore.

Another reason for rejecting the name, Equal-Annual-Payment Method, is that, as applied by the Committee, the name describes the the rate but not the depreciation allowance. The Committee, in its illustration on page 34, computes an amount, $3.02, as the first year's depreciation, and then states that if this amount is allowed for depreciation in each succeeding year there can be added to it each year 5% interest on the original investment, and the two together will provide a constant rate. This is an imaginary situation which will never exist after the first year, and the Committee then proceeds to explain that, although the depreciation will take place like a compound-interest calculation, yet the return on the remaining value of the property will follow it on down, so that the sum total will always be a constant rate. According to this, then, the compound-interest curve is the real theoretical depreciation line, and not the Equal-Annual-Payment line at all. Why, then, should not this method be called the Logarithmic-Curve Method? As the compound-interest curve is a logarithmic curve, this name will exactly describe the theory on which the depreciation is computed, whereas the Committee's name does not.

There is still another reason for getting away from the name, Equal-Annual-Payment Method. The Sinking-Fund Method calls for equal annual payments also, and thus the ambiguity arising from a confusion of the two will continue.

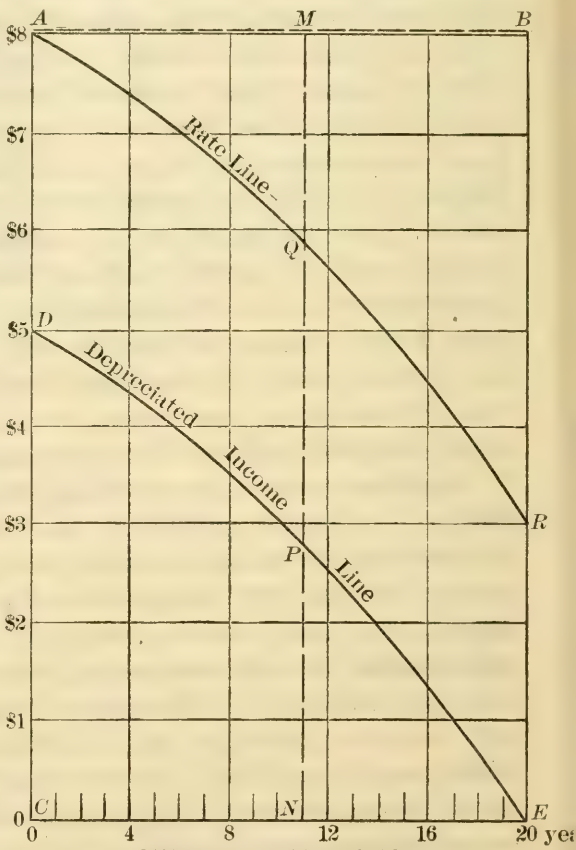
Mr. Crehore.

Logarithmic-Curve Method.—In Fig. 1 the Logarithmic-Curve Method is illustrated, all values being plotted from the Committee's table on page 34. The total rate, which is constant at value $8.02 (the top of the diagram) is made up of the two parts, one above, and the other below, the Depreciated-Income Line, *DPE*. This line follows the values given in Columns 3 and 4 of the table on page 34, so that, at any time during the life of the property, the ordinate, *MP* (above the curve), will represent the depreciation during the current



LOGARITHMIC-CURVE METHOD
SHOWING DISTRIBUTION OF THE RATE
THE COMMITTEE'S EQUAL-ANNUAL-PAYMENT METHOD

FIG. 1.



SINKING-FUND METHOD
SHOWING DISTRIBUTION OF THE RATE

FIG. 2.

year, and the ordinate, *PN* (below the curve), will represent the return on the remaining value of the property for the same year. This Logarithmic Curve, then, divides the rate at all times according to the theory advanced by the Committee for the treatment of depreciation. The Committee's imaginary line of equal annual payment is seen at *DF*, running horizontally.

Sinking-Fund Method.—The statement that the recommended method gives the same results as the so-called Sinking-Fund Method is only partly true so far as only the items of annual depreciation

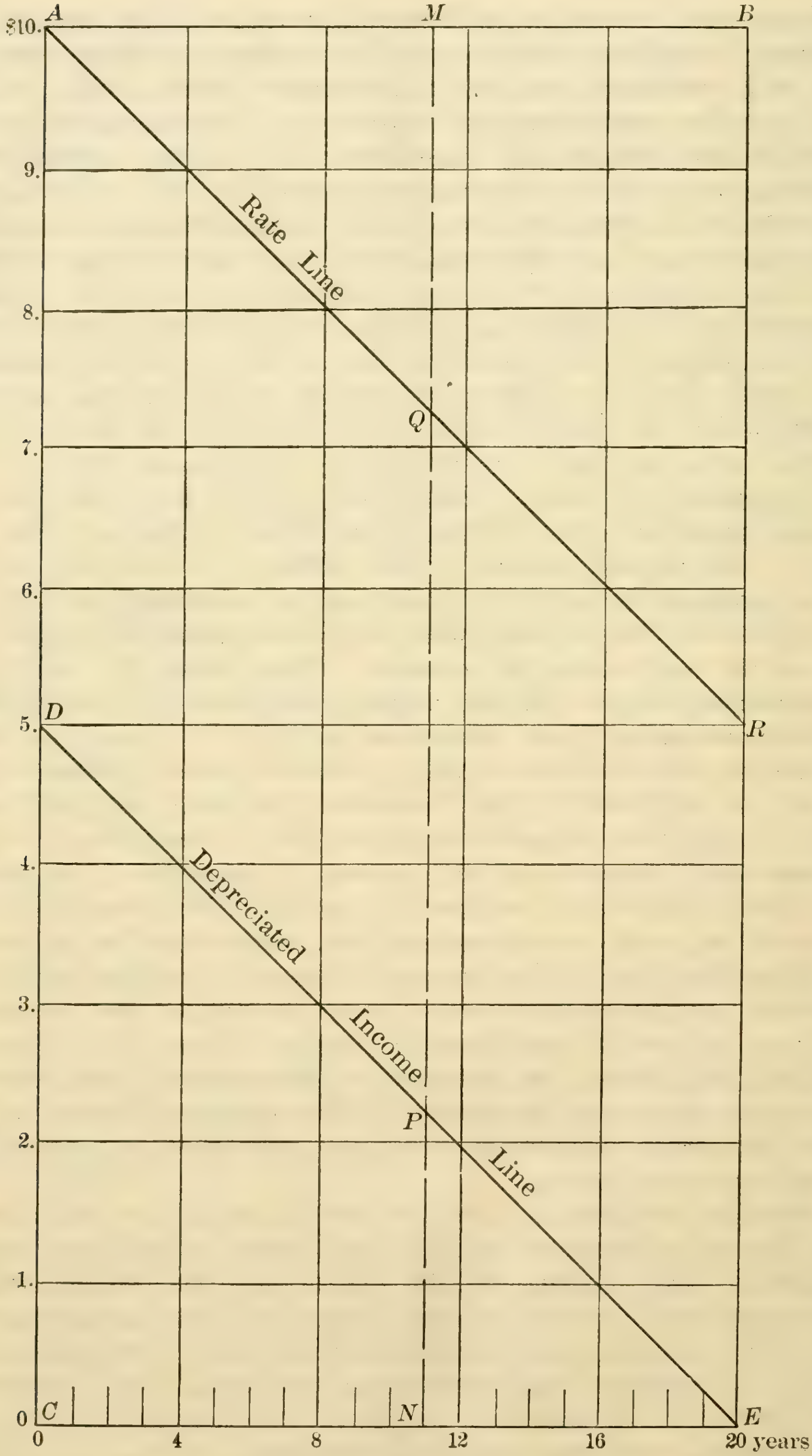
allowance and income on the remaining value of the property are concerned in making up the rate. Fig. 2 shows that these two items alone will produce a rate line following the curve, AQR, which is made up of a constant annual payment to the sinking fund, represented by any ordinate, QP, together with the annual return on the remaining value of the property, represented by PN. All values on this curve, AQR, then, are given by the sum of these two items at any particular time. This, therefore, constitutes the rate line unless we are to assume some other feature in addition to these two items. It is assumed by the Committee, however, that the rate line of the Sinking-Fund Method is the horizontal line at the top of the diagram at value $8.02. In order to bring the rate up to that line, the compound interest which the sinking fund is to earn has to be assumed to be included in the rate, and is represented by the ordinate, MQ, in Fig. 2. Mr. Crehore.

A comparison of Figs. 1 and 2 will explain why there has been so much opposition to the Sinking-Fund Method in the past. The advocates of this method demand a rate sufficient to cover the constant annual payment, QP or AD, in addition to a constant return on the original value of the property, which would be represented by the value, DC, all the way across the diagram. They claim the right to an undiminished income from the property because of the establishment of the compensating fund. Referring to Fig. 1, it is seen that this is just the way the Committee's Equal-Annual-Payment Method is made up, but, as explained by the Committee, the total rate paid carries with it, not only the annual contribution, MQ, to the sinking fund, but the compound interest on it, QP, as well. Opponents of the Sinking-Fund Method object to it because they say all contributions to the fund have to be set aside to earn interest, and cannot therefore be used for any thing else; but, if the compound interest is included in the rate paid each year, then this objection has no weight, as then the sinking fund may be used for any desirable purpose. Those who advocate the Sinking-Fund Method are mistaken in demanding a full return on the original investment in addition to the compound interest on their fund. The former includes the latter, and they cannot expect to charge for the same item twice.

Straight-Line Method.—For the sake of comparison, the Straight-Line Method is illustrated in Fig. 3, all values being taken from the Committee's table on page 40. It is plain that the rate line will fall away constantly, clear to the end of the life of the property, which means very much larger rates in the earlier than in the later years of its life, an unnatural and burdensome condition. The Logarithmic-Curve Method is the logical as well as the most equitable that has yet been proposed, and seems to meet all objections.

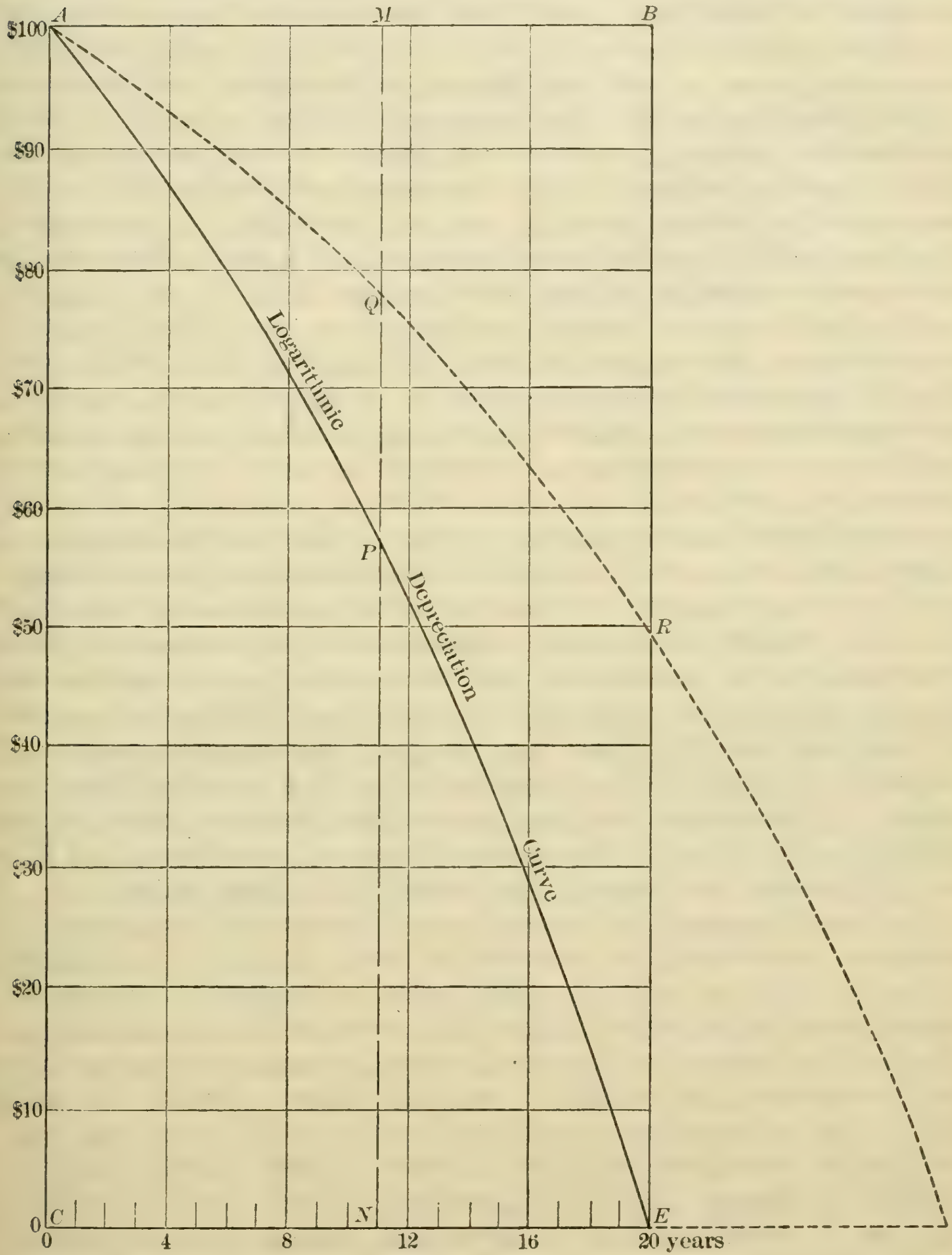
Logarithmic Curve of Depreciation.—In Fig. 4 is illustrated the curve of depreciation itself, based on the Logarithmic-Curve Method.

Mr. Crehore.



STRAIGHT-LINE METHOD
SHOWING DISTRIBUTION OF THE RATE
FIG. 3.

Mr. Crehore.



LOGARITHMIC-CURVE METHOD
SHOWING ASSUMED DEPRECIATION

FIG. 4.

Mr. Crehore. The upper horizontal line of the diagram, *AB*, represents the full value of the property at which the depreciation curve starts in the first year. If it be assumed that at the end of the property's life it will have no value, not even scrap value, then the curve of depreciation will terminate at zero at the end of life, as shown. Any ordinate to this depreciation curve, as *MP* from the top of the diagram, will represent the total depreciation since the beginning, and the ordinate, *PN*, will represent the remaining value of the property at the same time, the sum of these two ordinates always being the total original value of the property. The full import of the influence of repairs and renewals is here observable. Beyond a certain point, repairs and renewals will not only prevent depreciation, but will actually build up the property. Assume that at a given time, the eleventh year of its life, an amount is expended on the property to bring its value up from *PN* to *QN*. Then, if this degree of efficiency were to be adopted for the future a new logarithmic curve could be constructed showing that the property would have a very material value, *RE*, at the end of 20 years, or by extending the curve show an increase of life up to 30 years. Of course, this leaves out of consideration such items as functional depreciation or obsolescence. It is often a question of expediency with the management, just what degree of efficiency should be maintained in individual instances. Rapidly approaching obsolescence would determine in favor of allowing rapid depreciation, other things being equal; and yet the property while in use at all must be maintained sufficiently to enable it to perform its duty. There is no denying that extensive repairs and renewals will diminish the depreciation or the so-called "final renewals." It is evident from the diagram that whatever portion of the 100% valuation is not in the property belongs to depreciation, and if it were all put into the property there would be no depreciation.

In these diagrams the vertical scale is purposely exaggerated to emphasize more clearly the depreciation lines. Of course, the logarithmic curve is not the only line which might be assumed to separate the annual depreciation payments from the return on the remaining property, but the assumption of any other than a compound-interest curve has the manifest disadvantage of requiring either a variable rate to be paid or else of assuming an arbitrary return on the remaining property, not in accordance with any standard uniform interest rate, but varying and erratic. As to the logarithmic curve, it may be placed in any one of several positions by assuming different interest rates for each position, the position of no interest rate being of course the straight line itself. A curve of this sort is preferable to a straight line, for it is well known that most kinds of property depreciate more rapidly toward the end of life than near the beginning. The determination of the correct position of this line for a given item of property can and should be approximated by appraisals taken from time to time

or by knowledge previously gained from some similar item of property. Opponents of all theoretical methods of depreciation analysis deny their practicability, and claim that no theory can be made to govern the matter. It should be the endeavor of advocates of theoretical depreciation to adopt such a theory as will conform as closely as possible with practical results by appraisement. Mr. Crehore.

Payment of Depreciation a Return of Capital.—The Committee takes the strong position that these principal and interest payments, as depreciation allowances, are actual returns of capital to the corporation, and should be treated as such. These payments should be expended by the corporation only as capital is spent. Failure to observe this point has been the cause of many a financial catastrophe. Those having authority over public utilities should insist on this point, in order to prevent misleading the owners of the property as to their real income, as well as to protect the rate-payers from subsequent demands on them to supply, a second time, depreciation contributions which have previously found their way to the stockholders in the guise of dividends. If these payments have been properly reinvested as capital, then an inventory would be sure to include them, although the particular item of property, on which this depreciation money was paid to the corporation, might be appraised at its depreciated value. There could be no injustice in this transfer of capital expenditure from one asset to another. The injury always arises from the misuse of capital.

C. P. Howard, M. Am. Soc. C. E. (by letter).—The writer has read this very able report with great interest. As stated therein, the result is "a physical valuation, dealing with the cost of existing structures with little regard to their merit or efficiency", which, "is not by itself a satisfactory basis for rate-making". Other matters to be considered by rate-making bodies, such as favorable location, are briefly discussed, in regard to which the Committee "has not been able up to this time to formulate general rules". Mr. Howard.

The recommendation as to appreciation of land is a matter which should be considered most carefully before being accepted generally. The Committee quotes Commissioner Maltbie of the New York Public Service Commission, closing with these words:

" 'Why should these matters be considered less definite when applied to land than when applied to the buildings thereon? The depreciation of buildings is a charge against operation; why should not the appreciation of land be a credit?' "

The answer to this question is obvious: The funds provided for depreciation are a part of the general expense of running the business —maintenance, repairs, and replacements. Replacements are, in a way, simply maintenance and repairs on a larger scale, and at longer intervals. Depreciation funds take care of replacements.

Mr. Howard.

The appreciation of land is entirely different. It has no necessary and direct connection with operation, or with the special use made of any particular piece of land. In a general way, of course, all industries in the community contribute to it. All other owners of property enjoy this increase as a clear bonus; the unearned increment. Why should the investor in a railroad be selected as the one of all others who shall not receive it? It is proposed to take from the corporation this benefit.

If the land increases in value, the rates to the public are to be correspondingly reduced.

The citizens of the community will enjoy, not only the increase in value of their own lands, which increase was created in part by the railroad, but also any increase in the value of the railroad lands, which sum in effect is to be taken from the company and distributed among them.

The man who buys a farm reaps his crops, and takes the profits. Any general increase in the value of land is his also. Why should it be different with a railroad company?

The Committee quotes from the decision of the United States Supreme Court in the Consolidated Gas Company case, in part as follows:

"'If the property which legally enters into the consideration of the question of rates has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule.'"

If the corporation shall "in effect" pay "to the public the amount of the appreciation" as recommended by the Committee, what benefit will the company receive from such increase? Absolutely none.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.*

By Messrs. W. H. Fulweiler, Sanford E. Thompson, and Prévost Hubbard.

W. H. Fulweiler, Assoc. M. Am. Soc. C. E. (by letter).—The writer regrets his inability to be present to congratulate the Committee on the subject-matter and method by which it has collected and crystallized into concrete form so much of the recent development in the use of bituminous materials. Mr. Fulweiler.

The use of such materials has advanced so rapidly during the past few years that naturally differences of opinion have arisen regarding the many details that enter into their utilization in road construction and maintenance. Many of these points have been covered by the Committee in such a manner that the writer believes that the report will be of the greatest practical benefit to many engineers, who, though they have been in charge of road construction, have been unable, through press of other duties, to give this subject the attention that it deserves, and who, therefore, have been frequently led into erroneous opinions. Relative to this matter special attention should be called to the Committee's statement regarding its conviction that there is no such thing as a "Panacea" for all highway ills, as the writer believes that at the present time there is a tendency on the part of some road authorities to insist on the use of a single type of construction under any and all conditions, which tendency has failed, and will fail, in many cases to result in the most economic type of construction.

* This is a discussion on the Report of the Special Committee on Bituminous Materials for Road Construction, for 1913, presented at the Annual Meeting, January 21st, 1914.

Mr. Fulweiler. In connection with the use of the traffic census in deciding on the most economic type of construction, it is believed that more attention should be paid to the position of the proposed improved road in relation to the general system, and the effect of its improvement on the general direction of traffic.

For a number of years the writer has held the view, as expressed by the Committee, that the objectionable slipperiness of bituminous surfaces can be decreased or prevented by proper precautions during construction, and that the succeeding paragraph in the Committee's report points out one of the principal precautions to be observed, namely, to reduce the crown on roads constructed with bituminous materials to less than ½ in. per ft.

The protection of the edges of bituminous pavements is very important, especially in view of the fact that, where there is a considerable proportion of high-speed motor traffic, many of the older roads are so narrow that they do not permit sufficient clearance without the general use of the shoulder in passing. This point, however, is already beginning to receive attention in several States.

It is believed that the Committee might have dwelt even more strongly on the fact that the character of the stone must influence the choice or grade of the bituminous material to be used, and that, after all, the stone must eventually carry the traffic, and, though cementitious qualities in the stone are no longer necessary, the other desirable qualities of hardness, toughness, and low percentage of wear, seem quite as essential as in stone for water-bound construction.

The Committee's remarks on the maximum amount of distillate up to 170° cent. allowable in tars, reminds the writer of his professor in mineralogy, who, in his lectures, would, in describing the color of a mineral, proceed somewhat as follows: "The color is generally blue, frequently green, with yellow varieties quite common. Many red specimens have been found at Franklin, N. J., and a number of mines in Colorado produce quantities of black twins, while a translucent variety is widely distributed through portions of Pennsylvania."

It is especially gratifying to the writer to note that the Committee has set forth so clearly the necessity for care in the use of bituminous materials, in the paragraph, where it is stated:

"Whatever method of construction is selected, the use of a bituminous material by no means justifies any lack of care in the ordinary details to be followed, but rather increases the need for thoroughness and skilled supervision."

The writer believes that probably 70% of the failures which have been attributed to the bituminous material have been due, as suggested by the Committee, to lack of care in details and supervision.

The writer cannot agree with the Committee in reference to the question of the use or omission of a seal coat on bituminous pavements, as he believes that the seal coat, if for no other purpose than

insurance against incidental imperfection or carelessness in the surface treatment is always advisable, and that the sole cause for its omission should be lack of funds. Mr. Fulweiler.

The writer wishes to emphasize the thought of the Committee on page 4, where it is stated that: "Where the character of the traffic justifies the use of the bituminous concrete pavement, the same conditions demand an extraordinarily strong foundation therefor." Heretofore, it is believed that too little attention has been given to the question of foundations and their effect on the life of the surface.

The writer most strongly agrees with the Committee that hand-mixing methods should be avoided where practicable, and that, in the use of a heated aggregate in the construction of a bituminous concrete pavement, non-uniformity or excess in heating the stone should also be avoided. This practically recommends the use of some mechanical form of dryer and heater, which the writer believes to be essential for uniformly successful results.

In the Construction by Penetration Method, it is believed that an equally important factor for successful results is a thorough compaction of the road metal before the application of the bituminous material, secured by proper grading of the road metal. The writer believes that compaction will never be secured with stone of uniform size until a certain proportion has been crushed into chips, which, under the action of the roller, serve to key or lock the larger pieces together, and produce that mechanical interlocking which is necessary to stability. He also believes that this same stability can be secured with very much less rolling by the use of stone of varying sizes down to, say, $\frac{3}{4}$ or $\frac{1}{2}$ in., and that such a stone mixture can be compacted with very much less rolling, thus resulting in a cleaner stone surface, and, therefore, better penetration and adhesion of the bituminous material.

In Construction-Surface Treatments, with which type the writer has probably had the greatest experience, he would suggest that the first paragraph read:

"The success of surface applications is proportionate to the penetration obtained by the bituminous material into the old road surface. and that such degree of cleanliness of the latter must be had, by sweeping or other methods, as will ensure the proper degree of penetration."

As the result of his experience, which would seem to indicate that penetration into the surface of the road is essential to success, surface treatment depending entirely on surface adhesion, even if it be perfect, will disintegrate by the failure of the water-bonding of the crust beneath the bituminous treatment.

From a careful study of a large number of roads, it has seemed to the writer that the most successful results have been attained where the penetration into the surface varied between $\frac{1}{2}$ in. and 1 in. A

Mr. Fulweiler. deeper penetration ($1\frac{1}{4}$ to $1\frac{1}{2}$ in.) was frequently indicative of too fluid a material, with the resulting deficiency in binding or bonding strength.

Too much stress cannot be laid on the cleanliness of the surface, as this is a fundamental requirement in successful surface treatment. The writer is also quite in accord with the Committee in disapproving of the use of fine sand, and in approving of the use of clean stone chips or small gravel for surface applications, and also in strongly disapproving of any excess in material.

He does not believe that under normal conditions the application of a bituminous material on a water-bound or bituminous surface is justified, unless it is covered with a mineral aggregate, as the cost of the latter is rarely 40% of the total cost of treatment, and he believes, therefore, that the rôle of the bituminous material in maintenance work is secondary to the stone, or rather mineral aggregate, and that its primary object is to protect the stone and assist in maintaining it in such a position that it may most efficiently and economically carry the traffic.

The next paragraph he believes should have the widest publicity, especially among many engineers to whom the care of the roads is but part of their work, and that is, that "the success of results depends first upon the condition of the surface to which the treatment is applied. Even the best surface treatment cannot correct defects in the original surface, though it may reduce or modify them."

Many cases have come to the writer's attention in which an improperly constructed, water-bound road had been given a light surface treatment with the expectation that such treatment would correct all the inherent defects of the original construction.

It is believed that the results obtained during the last two or three years would have justified the Committee in expressing its opinion on pressure distribution more strongly than it has. The writer's opinion, based on some four years' work with pressure distributors, and six years' observation of gravity distribution, is that with the use of pressure distributors, from 20 to 25% of material may be saved, 50% better penetration may be secured, and the over-all efficiency, as compared with gravity distribution, is about as two is to one.

So many discussions have recently appeared in the technical journals regarding the question of definitions, that the writer is rather loath to mention the subject, but he would call attention to the Committee's definition of "free carbon," and would suggest that "organic" be inserted before "matter," as bringing this definition somewhat more in line with the methods used for the determination of this constituent. It is not believed that the use of the term "free carbon" can be defended to any very great extent, as the material is not pure carbon, and the quantity varies with the method of determination and the solvent used. The writer believes it to be better practice to

use the term "material insoluble in," or "organic material insoluble in carbon bisulphide," without attempting to give a name known to be erroneous, to a compound or mixture of varying composition. Mr. Fulweiler.

Under "water-gas tars," the writer does not believe that the word "cracking" is used correctly in this connection, but that a more correct definition would be: "tars produced by the decomposition and polarization of oil vapors in the manufacture of carburetted water gas." The same criticism might be applied to oil-gas tar.

There are several points in connection with the report which the writer would like to discuss, but which he feels he should refrain from doing, as it would involve the discussion of characteristics that serve to classify certain commercial materials, and he does not believe that such a discussion by one interested in the commercial development of bituminous materials would generally be considered as unbiased.

Sanford E. Thompson, M. Am. Soc. C. E. (by letter).—In this report the writer notices that "aggregate" is defined as "The mineral portion or the larger sized particles forming the body of a concrete or of a road surfacing," etc. Mr. Thompson.

This definition is at variance with the use of the term in connection with concrete construction, and as universally adopted in practice. It also disagrees with its use by the Special Committee on Concrete and Reinforced Concrete.

The generally adopted use of the term, "aggregate," is: the entire mineral inert material, such as sand, broken stone, etc., in a concrete or a pavement.

Any other definition is illogical and unscientific, because there is no well-defined line between fine and coarse aggregates, and, in certain cases, the coarse aggregate may be a material no coarser than a very coarse sand. Even in a bituminous mixture, there is no line of demarcation after the fine and coarse materials are in place in the pavement, the effect of the mixture being simply a graded mineral substance ranging from coarse to fine. The writer, therefore, offers the following as a substitution for the definition of aggregate:

Aggregate.—The mineral inert material, such as sand, gravel, and broken stone, with which the cement or the bituminous material is mixed to form a cement concrete or a bituminous concrete. Fine aggregate may be considered as the mineral inert material finer than $\frac{1}{4}$-in. mesh in size and coarse aggregate the material coarser than $\frac{1}{4}$-in.

Prévost Hubbard, Assoc. Am. Soc. C. E. (by letter).—The following comments are made with some reluctance on the part of the writer because for a number of years he has been engaged in an attempt to standardize the nomenclature of bituminous materials through his connection with a sub-committee of six engineers and chemists Mr. Hubbard.

Mr. Hubbard.

The recommended definition for "Fixed Carbon" differs from that adopted by the American Society for Testing Materials, for no apparently good reason, and is, in fact, incorrect. The word "coke," after residual, has been left out, and the words "highly heating" have been substituted for "burning". Thus the adjective "residual" has been made to take the place of a noun. If the word "coke" is eliminated, "residual" should be changed to "residue" or "residuum". Even if this were done, however, the definition would be incorrect, as the process indicated might be conducted on a bitumen without producing fixed carbon. It is quite possible to heat asphaltic hydro-carbon products highly in the absence of free oxygen without producing coke, in which case fixed carbon would not be produced. If, however, the material is burned in the process, coke is produced and also fixed carbon.

The recommended definition for "Flux", though correct as far as it goes, does not go far enough. For example, fluxes are limited to "fluid" and "semi-fluid bitumens", whereas they may be solid products. Thus naphthalene, a white, crystalline solid, may serve as a flux for hard tar-pitch, a combination of the two producing a soft or even fluid bitumen.

In view of the foregoing it is suggested that the proposed definitions be given very careful consideration with a view of possible modification before they are accepted by the Society.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS.*

By Messrs. Edgar Marburg, P. A. Beatty, H. E. Phelps, Ernest McCullough, M. J. Henoch, and Leonard Lundgren.

Edgar Marburg, M. Am. Soc. C. E. (by letter).—The writer has read this report with interest. Inasmuch as it is the intention of the Committee to follow this with a more complete report, the writer ventures to suggest the desirability of considering the preparation of two additional average yearly compensation curves, of which one would exclude the upper and lower 5% of the returns for each group, and the other the upper and lower 10% for each group. These curves would thus represent, respectively, the average compensation of the middle 90 and 80% of the membership of each group, and, as such, would serve as more reliable criteria as to what may be called the "average salary expectation curve," according to length of professional service, in so far, at least, as such expectations can be safely premised on experiences. Mr. Marburg.

To illustrate the misleading character of the present average yearly compensation curve, attention is called, for example, to the group of 69 members who have had 27 years of experience. In that group one particular individual happens to command the extraordinary professional income of $150 000 per annum. If that individual alone were excluded from this group, the average yearly compensation of that group would be reduced by more than $2 000. In the other group, in which a single income of $150 000 appears, there are 118 members, and despite this relatively large number, the exclusion of this one individual from that group would reduce the average yearly compensation of the entire group from $5 721 to less than $4 500.

*This is a discussion on the Report of the Special Committee to Investigate Conditions of Employment of, and Compensation of, Civil Engineers, presented to the Annual Meeting, January 21st, 1914.

Mr. Marburg. The writer is strongly of the opinion, too, that the earnings of the 3 214 members who failed to respond to the circular inquiry are likely to be very much lower than those of the 3 636 respondents. The effect of the exclusion of this silent element on the returns reported by the Committee must necessarily remain a matter of conjecture, but, in the writer's judgment, it can hardly be doubted that its effect, if it could be determined, would be decidedly in the direction indicated. It is only human that an individual who realizes that he has been more or less of a failure, at least by the standard of earnings, would be reticent in his attitude toward an inquiry of this nature, notwithstanding assurances of secrecy.

The writer shares the opinion commonly held by members of the Engineering Profession, that, as a professional class, engineers are underpaid. In the broad consideration of that question, the average compensation curve is much more significant than either the maximum or minimum compensation curve. It seems especially important, therefore, that the average yearly compensation curve should also be constructed in the manner suggested, in order that the danger of drawing hasty and unfairly optimistic conclusions from the data collated by the Committee may be minimized; for even then, as has been pointed out, the showing would probably be much more favorable than is to be anticipated from complete returns from the membership of the Society, not to mention the large element—especially in the lower professional ranks—which is not identified with the Society.

Mr. Beatty. P. A. Beatty, M. Am. Soc. C. E. (by letter).—The writer would suggest that it might be useful to prepare an "average yearly compensation" line of those who receive $15 000 per annum or less from the practice of their profession.

The percentage of engineers who receive more than $15 000 annually is very small, the writer believes, and the elimination of those who receive more would lower the "average yearly compensation" line considerably. For example, take the twenty-seventh year. The average yearly income of 69 engineers is given as $8 023. Eliminating the $15 000 man, the average annual income of 68 engineers is reduced to $5 935.

To obtain an idea of, say, 85% of the Profession, which would probably be useful in an investigation of this kind, the elimination of exceptional incomes would give a line representing the earning capacity of the greater part of those engaged therein.

Mr. Phelps. H. E. Phelps, Assoc. M. Am. Soc. C. E. (by letter).—This report is most interesting and valuable. However, it does not seem probable that the average yearly compensation curve, as determined by the Committee, correctly represents actual conditions. The fact that only 53% of the membership of the Society reported their compensation, is

doubtless due to a variety of causes, but it seems reasonable to believe that the main cause is an unwillingness to report a low compensation. Moreover, if the words "Civil Engineers" refer—as generally assumed—to all graduates of technical schools and all others with an equivalent education and training, and not only to members of the American Society of Civil Engineers, the compensation of these men should be ascertained, if possible, and incorporated with the information already found, in order to show the true state of affairs. Mr. Phelps.

It is doubtful if the well-nigh universal dissatisfaction of engineers with their income and general status would be so pronounced if the curve presented by the Committee showed the true state of affairs, low though it is. From the writer's acquaintance among engineers, he does not believe that their average yearly compensation is one-half, or, at most, two-thirds, of that shown. Doubtless, many engineering firms are represented therein by the large incomes of the members of such firms, and not at all by the small incomes of the draftsmen, transitmen, and assistant engineers employed by them. The members of this Society should not spend too much time contemplating the dizzy altitudes reached by the maximum compensation curve, but rather should study how to raise both the true minimum and average curves, which are much too close together and lower than they should be. The Society itself should take some action along this line, and the writer believes the most effective action would be, in some way, to assist engineers to obtain employment when they need it.

ERNEST McCULLOUGH, M. AM. SOC. C. E. (by letter).—The writer has read this report with interest, and feels certain that it will be ably discussed by the editors of the leading technical journals. He hopes that these editorials may be offered by members of the Society and be printed in the *Proceedings* and *Transactions*, for he is sure that the opinions of the editors will be a reflection of those of many members. Mr. McCullough

The report is based on replies from about one-half the membership of this Society. The total number of engineers enrolled in the four national engineering societies—Civil, Electrical, Mechanical, and Mining—cannot be more than 20 000, when one takes into consideration the number of men enrolled as members in more than one of these societies, the total of the separate lists reaching to more than 27 000. These represent the men who have achieved some measure of success, yet many among them feel that Fate has not been kind. Of the 20 000 men, less than 4 000 have answered in such a manner that their replies could be incorporated in the report.

Statistics from engineering schools show that the number of such schools has increased greatly in the past 20 years, and that of the total number of men graduated from such schools, more than 60%

Mr. McCullough. have been graduated since 1900, just 14 years ago. The first school of engineering was established in the United States in 1824, and the first man was graduated in 1826. The increase of interest in technical education, therefore, has exceeded the increase in population, but as engineering is a profession which is peculiarly the product of civilization, the increase, perhaps, is not disproportionate to the general world advance.

Numbers of men calling themselves engineers, some of whom attain high standing, are non-graduates, and many are engaged in engineering work who did not complete a four-year course in a technical school. That numbers go into the work after partly completing a course of study in engineering schools is well known, and many of these men are good engineers. The writer has been informed by a professor that on an average probably 20% of the freshmen complete the work of the senior class. The number of alumni of a school, when compared with the graduates, shows that this statement cannot be far wrong. In a certain college in the United States—not an engineering school, however—established in 1837, the number of graduates has been 15 000, yet the living alumni number 12 000, a majority being enrolled as members of the alumni association.

This being admitted as a state of affairs existing in engineering schools, the number of men practicing engineering in the United States must be several times the number enrolled in the national societies. It may be not far out of the way to state that the less than 4 000 men whose reports are tabulated and shown on the compensation chart issued by the Committee, is a very small percentage of those whose interest in the work of such a committee is lively.

In the writer's opinion, it is not so much the matter of compensation that the Committee should consider as the conditions of employment and the treatment of the men who have received enough training in engineering to lead them to follow it as a vocation. The custom is to engage them in a hurry in order to push work to completion, and rarely is a man given a week's notice of dismissal. The majority of men employing engineers and engineering assistants prefer to keep them in ignorance as to when their services will be no longer required, fearing that they will leave for a new job before their work is completed, thus placing the employers under the necessity of getting new men and thereby increasing the cost for engineering services. It is common with railways to pay their engineers from the 20th of the month up to the 1st, thus retaining two-thirds of a month's pay. Frequently, the men are dismissed on the 1st and their pay is not forwarded to them until the regular time, giving them no surplus on which to live until it comes or a new job is secured. It is not uncommon for men to be discharged after many years of faithful service, and be replaced by younger men at lower pay.

The seasonal employment of engineers, so far as this factor relates to their work, puts them on a plane with common labor. The pay being intermittent and the men being subject to the same troubles that all are subject to, such as illness in their families, injuries and illness to themselves, sporadic employment at pay which is seldom more than a bare living wage when employed, being below a living wage when the average is considered, causes much needless distress in a class of men whose training must be high and who are usually well educated. The work of the engineering societies should be directed toward securing better terms for all men who have received the necessary training. Mr. McCullough.

Numerous companies make a good profit in securing employment for technically trained men. Their rates are very high, and they enforce collections mercilessly. Men are given positions—which these companies say will be permanent—only to lose them in a short time, often before the commission to the employment agency has been fully paid.

The Society should start some method, in connection with the other national societies, whereby employment may be obtained, a clearing house as it were, for the placing of jobless men in touch with jobs. Members of the societies should pledge themselves to employ only men recommended to them by this department. The Committee, further, should enlarge the scope of the inquiry to cover every man who has been graduated from an engineering school within the past twenty-five years. This would imply that the inquiry into conditions existing among engineers should be extended to all engineering graduates, and not to men who are not graduates. The men, however, who have achieved such a measure of success as to enable them to be enrolled in the American Society of Civil Engineers should be treated as a distinct class. The writer knows a very large number of men fully qualified to be members of this Society who do not join for the sole reason that their income is so precarious, and who are out of work so often, that they feel that it is impossible to ally themselves with the Society. These are the men who really impress employers with the fact that many things are not as they should be with the vocation of the engineer.

M. J. Henoch, M. Am. Soc. C. E. (by letter).—The writer is very much interested in this report. He realizes the great amount of time and labor required to collect, compile, and digest the mass of data requisite for the proper consideration of this important subject, and it may be, therefore, that the following remarks are premature. They are not offered, however, with any intent to criticize the excellent work of the Committee, but to bring out two points which appeal to the writer. Mr. Henoch.

The first is in regard to the Committee's conclusion that the membership of the Society is large enough to be representative, and to be typical of the various stages of engineering life.

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The first is in regard to the Committee's conclusion that the membership of the Society is large enough to be representative, and to be typical of the various stages of engineering life.

The members of this Committee are all eminent and successful engineers, live in an environment of "big things", and very naturally view conditions from a different standpoint from those who are not so successful. The writer has no means of knowing what percentage of those actually engaged in the practice of civil engineering in the United States are connected with this Society, but the proportion certainly is small.

The membership of the Society is chosen by a process of elimination which excludes the less successful engineers, and there are thousands who, although eligible to membership under the rules of the Society, are, nevertheless, unable or unwilling to afford the expense thereof, small as it may appear to their more prosperous brethren. There are many trained and capable engineers working for salaries of from $125 to $150 per month, who have families or dependent relatives to support, and to them the payment of $25 for entrance fees and $15 for annual dues looks like an insurmountable obstacle. It may be said, therefore, that, as a rule, it is only the more successful engineers who become members of the American Society of Civil Engineers, that the membership is not large enough to be representative, and that it is not typical of the various stages of engineering life.

Also, in regard to the curve of average yearly compensation shown on the diagram presented by the Committee, it appears to the writer that this does not show the thing which appeals to the average engineer. It is true that it shows the average yearly compensation of the engineers reporting, but it does not show the yearly compensation of the average engineer. An inspection will show that this curve is unduly influenced by the curve of maximum compensation, and that every fluctuation of that maximum curve is reflected by a corresponding fluctuation in the average curve, but the minimum curve has, in general, no corresponding effect.

It will generally be admitted that the very high salaries reported are exceptional, and are by no means as frequent as the low salaries, yet their effect on the average curve, as compared with that of the low salaries, is (roughly speaking) inversely proportional to their relative frequency. For instance, the conditions shown for 19 years' experience is an exaggerated case, but serves well as an illustration. In this case the maximum salary is 114 times the minimum, though the average is only 4.3 times the minimum. Considering the extremes only, it requires 32.74 men at the minimum salary with 1 man at the maximum salary to produce the average shown; or, in other words, it requires practically 33 men at the minimum salary to offset the effect of 1 man at the maximum salary. The ratio of the minimum to the average is 1 to 4.33, though the ratio of the average to the maximum is 1 to 26.33.

It would appear, therefore, that the thing which really shows the conditions of the average engineer is (to use the terms of the Committee) the salary of the "middle-man" or that compensation which is exceeded by and which exceeds that of an equal number of men.

LEONARD LUNDGREN, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has studied this report with interest.

The Committee states that the average curve shown in the report is, if anything, too low; that the general average of compensation of [illegible] members is $[illegible]; and that the average yearly compensation for [illegible] members whose experience is 10 years or less is $[illegible].

The writer feels that the impression will be created that the average engineer receives the average yearly compensation shown by the diagram accompanying the report. The informed know that this is not the case, and that the majority of those answering the questions of the Society were the most successful engineers in the United States, which fact raised the average considerably. This point should have been emphasized in the report. If the curve was intended to show the ideal salary that the Committee thought an average engineer should be paid, it would be admirable.

It is known that the minimum wage of a technical graduate of one year's experience is $[illegible] per year; that $[illegible] per year is considered a good job; and that $1 [illegible] per year is an exceptionally good position; and the writer can scarcely believe that an engineer would receive from any one the maximum salaries given in the report ($2 [illegible] for one year's experience, and $5 000 for two), except for reasons outside of the realm of engineering.

The conditions after 20 years' experience can be analyzed similarly. The lowest wage known to the writer for this term of service is $[illegible]. An engineer is considered by his co-laborers as a success if he receives a salary of $[illegible], and as very successful if he receives a salary of $[illegible]. The average shown by the diagram is $4 [illegible], which is at least [illegible]% higher than the salary of the average engineer of 20 years' experience.

The writer quite appreciates the position of the Committee as to lack of data, but he feels that it has overlooked numerous possible sources of information, such as the salary lists of the engineering branches of the United States Government, of the States and large cities, of the railroads, and of the large manufacturing corporations, many of which could have been secured by proper requests.

It was hoped that the Committee would secure sufficient information to produce a diagram, similar to that accompanying the report which, in addition to the minimum and maximum curves, would have a curve showing what salary an average engineer would receive for his working life. This it is felt has not been accomplished.

Mr. Hoch. The members of this Committee are all eminent and successful engineers, live in an environment of "big things", and very naturally view conditions from a different standpoint from those who are not so successful. The writer has no means of knowing what percentage of those actually engaged in the practice of civil engineering in the United States are connected with this Society, but the proportion certainly is small.

The membership of the Society is chosen by a process of elimination which excludes the less successful engineers, and there are thousands who, although eligible to membership under the rules of the Society, are, nevertheless, unable or unwilling to afford the expense thereof, small as it may appear to their more prosperous brethren. There are many trained and capable engineers working for salaries of from $125 to $150 per month, who have families or dependent relatives to support, and to them the payment of $25 for entrance fees and $15 for annual dues looks like an unsurmountable obstacle. It may be said, therefore, that, as a rule, it is only the more successful engineers who become members of the American Society of Civil Engineers, that the membership is not large enough to be representative, and that it is not typical of the various stages of engineering life.

Also, in regard to the curve of average yearly compensation shown on the diagram presented by the Committee: it appears to the writer that this does not show the thing which appeals to the average engineer. It is true that it shows the average yearly compensation of the engineers reporting, but it does not show the yearly compensation of the average engineer. An inspection will show that this curve is unduly influenced by the curve of maximum compensation, and that every fluctuation of this maximum curve is reflected by a corresponding fluctuation in the average curve, but the minimum curve has, in general, no corresponding effect.

It will generally be admitted that the very high salaries reported are exceptional, and are by no means as frequent as the low salaries, yet their effect on the average curve, as compared with that of the low salaries, is (roughly speaking) inversely proportional to their relative frequency. For instance, the conditions shown for 18 years' experience is an exaggerated case, but serves well as an illustration. In this case the maximum salary is 114 times the minimum, though the average is only 4.3 times the minimum. Considering the extremes only, it requires 32.74 men at the minimum salary with 1 man at the maximum salary to produce the average shown; or, in other words, it requires practically 33 men at the minimum salary to offset the effect of 1 man at the maximum salary. The ratio of the minimum to the average is 1 to 4.33, though the ratio of the average to the maximum is 1 to 26.22.

It would appear, therefore, that the thing which really shows the conditions of the average engineer is (to use the terms of the Committee) the salary of the "middle-man" or that compensation which is exceeded by and which exceeds that of an equal number of men. Mr. Henoch.

Leonard Lundgren, Assoc. M. Am. Soc. C. E. (by letter).—The writer has studied this report with interest. Mr. Lundgren.

The Committee states that the average curve shown in the report is, if anything, too low; that the general average of compensation of 3 636 members is $4 325; and that the average yearly compensation for 1 972 members whose experience is 15 years or less, is $2 827.

The writer feels that the impression will be created that the average engineer receives the average yearly compensation shown by the diagram accompanying the report. The initiated know that this is not the case, and that the majority of those answering the questions of the Society were the most successful engineers in the United States, which fact raised the average considerably. This point should have been emphasized in the report. If the curve was intended to show the ideal salary that the Committee thought an average engineer should be paid, it would be admirable.

It is known that the minimum wage of a technical graduate of one year's experience is $360 per year; that $720 per year is considered a good job; and that $1 200 per year is an exceptionally good position, and the writer can scarcely believe that an engineer would receive from any one the maximum salaries given in the report ($2 000 for one year's experience, and $5 000 for two), except for reasons outside of the realm of engineering.

The conditions after 20 years' experience can be analyzed similarly. The lowest wage known to the writer for this term of service is $900. An engineer is considered by his co-laborers as a success if he receives a salary of $3 000, and as very successful if he receives a salary of $5 000. The average shown by the diagram is $4 941, which is at least 50% higher than the salary of the average engineer of 20 years' experience.

The writer quite appreciates the position of the Committee as to lack of data, but he feels that it has overlooked numerous possible sources of information, such as the salary lists of the engineering branches of the United States Government, of the States and large cities, of the railroads, and of the large manufacturing corporations, many of which could have been secured by proper requests.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion.*

By Messrs. Willis Whited, William H. Connell, E. W. James, Charles J. Bennett, A. W. Dean, Henry W. Durham, Robert A. Meeker, Paul D. Sargent, W. W. Crosby, P. E. Green, E. H. Thomes, S. Whinery, Mark Brooke, William R. Farrington, H. B. Pullar, Arthur H. Blanchard, W. H. Fulweiler, and James H. Sturdevant.

ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

Willis Whited, M. Am. Soc. C. E. (by letter).—This subject is of vital importance, because incompetency, inefficiency, or dishonesty in the handling of highway work may cause a larger percentage of waste of funds than in almost any other class of work. However excellent the personnel of the organization may be, there will be much waste unless all parties work harmoniously, steadily, and efficiently toward the end in view. There is the further difficulty, that, if the organization is not reasonably harmonious, many of the best men will become disgusted and resign, which would make it doubly hard to secure others to take their places. The subject can perhaps best be studied by taking a concrete example, such as the Pennsylvania State Highway Department. Mr. White

The Department was first organized by an Act of the Legislature in 1903. The Act, however, was amended from time to time until 1911, when an entirely new one was passed, known as the "Sproul Act", reorganizing the whole Department, so that its functions may now be divided into three classes.

(1) The Department, at the expense of the State, is to improve and maintain, as funds become available, about 8 800 miles of roads, mostly

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* At meetings held January 23d and 24th, 1914. As much of this discussion as possible is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

main thoroughfares, set aside by the 1911 Legislature, to which the Legislature of 1913 added about 1 200 miles.

(2) The Department, under the law, is required to furnish to counties, townships, and boroughs, as funds become available, State aid for the improvement and maintenance of highways. The State pays one-half of the expense, the other half being borne equally by the county and township, or county and borough, as the case may be, the Department having entire charge of such road work.

(3) The Department, under a Legislative Act of 1913, has technical jurisdiction over all improvements on about 82 000 miles of road in addition to the 10 000 miles previously mentioned, and these roads are under the control of the Township Road Supervisors acting in their respective townships. The Department also has charge of the purchase of road-building machinery, exceeding a certain amount. Contracts pertaining to work of this class are not valid unless approved by the Department. The State bears one-third of the expense of such improvements. This places all public roads, outside of cities and boroughs, under the more or less complete control of the Department.

For handling such extensive enterprises, involving the expenditure of many millions of dollars annually, three methods may be suggested:

(1) The appointment of a commission consisting of three or more members, who shall formulate all general policies and appoint officers to carry them out;

(2) The appointment of a single head, with advisory council;

(3) The appointment of a single head, without advisory council, except such as he chooses to accept from his subordinates for special purposes, the Legislature itself determining the general policy of the Department.

The advantage of the first method is that the united knowledge and experience of the whole commission is available for the settlement of disputed points and questions of policy. The principal disadvantages are: first, the division of responsibility; second, it is apt to become a mere debating club, which seldom accomplishes much unless dominated by one man who, being an official, can with difficulty be prevented from abuse of power. A third disadvantage of this method is that a competent commission for such a purpose is usually composed of very busy men, who have many other interests, making it extremely difficult to get them together with sufficient regularity to settle questions with the promptitude necessary for the most effective prosecution of the work, particularly as their salaries are apt to be much less than the real value of their services.

The principal advantages of the second method are that the members of an advisory council usually have an adequate salary, and can be required to hold meetings regularly. On the other hand, being

subordinates, they will have little power, and will therefore take but little interest in the work. Mr. White

The third method, in which all authority is vested in a single head, constitutes in reality a despotic rule, which is by far the most efficient provided the proper man can be placed in the position and prevented from any misfeasance of office. The superiority of this form is shown by the fact that all military organizations from time immemorial have been under the control of single heads having despotic power, except that in modern warfare civil authorities have the right to deprive him of his commission in case he should abuse his powers or show incompetency. In warfare, efficiency is of paramount importance. Almost all industrial and business concerns are under the control of single heads, but with powers subject to reasonable limitations. The Pennsylvania State Highway Department is placed under a single head, whose powers are limited by the ordinary laws of the Commonwealth and by the Act under which he is appointed, which designates the policy to be pursued by the Department.

All contracts are approved first by the Highway Commissioner and then by the Governor and Attorney General. All appointments to the higher positions are approved by the Governor. All expenditures are audited and approved by the Auditor General. Such checks as these should be sufficient to prevent the head of the Department from misusing his authority in making appointments and from the misappropriation of any funds.

Under the above-mentioned Act, the Commissioner is in absolute control of the administration of his Department, assisted by two Deputy Commissioners, also under his control.

The First Deputy Commissioner, in addition to assuming the duties of the Commissioner during his absence, is in charge of the Bureau of Township Highways, and also of the granting of permits for special uses of all public roads outside of municipalities.

The Second Deputy Commissioner, in addition to acting as an assistant to the Commissioner, has charge of the licensing of motor vehicles, and the enforcement of State laws relative to the use of such vehicles on State roads. He also has charge of the finances of the Department.

The Chief Engineer has immediate technical and executive control over the whole Department excepting the Automobile Division.

There is a clerical and auditing staff at headquarters, performing the duties usual with such positions.

There is an Engineer of Bridges, who has technical charge of all bridges and culverts.

There is a Superintendent of Signs, and a Superintendent of Asphalt Construction, whose duties need no description. There is also a Division of Tests, and a Division of Statistics, charged with the duty

of obtaining data as to the traffic on all roads under the jurisdiction of the Department.

There are fourteen Assistant Engineers, who have technical and executive control over their respective Districts, but who act only in a technical capacity on township road work, and an additional Assistant Engineer acting as office assistant to the Chief Engineer.

The maintenance of State roads is in charge of a Maintenance Engineer, subordinate to the Chief Engineer, and in charge of fifty Superintendents, each of whom has charge of the maintenance of all roads in his respective district. These Superintendents, acting under the direction of the First Deputy Commissioner, also have immediate technical charge of township roads and bridges, and are required to approve all plans for road improvements to be carried out by the Township Supervisors, of whom there are about 6 500.

All bills are approved by the various heads under whose jurisdiction they have been incurred, and are then audited by the Department Auditor and the Auditor General of the State, thus rendering impossible any considerable misappropriation of funds. All payments on contracts must be approved by the Assistant Engineer, the Chief Engineer, the Department Auditor, the Commissioner, and the Auditor General of the Commonwealth.

All surveys and plans are made by the staffs of the Assistant Engineers, and checked by the Chief Draftsman and his staff, located at headquarters. They are then approved by the proper Assistant Engineer, the Chief Engineer, and the Commissioner. Plans for bridges and culverts are made and checked in the Bridge Division, and approved by the Engineer of Bridges and the Chief Engineer.

Thus it will be seen that all matters of importance are passed upon by at least two competent men, who are entirely familiar with the conditions surrounding each case.

All contracts are approved by at least three men, in addition to the Attorney General of the Commonwealth, and all payments on contracts are checked and approved by at least three men before finally being approved by the Auditor General.

All work done under contract is under the direct supervision of an Inspector on the work, who is thoroughly conversant with the details. Such work is approved by the Assistant Engineer and inspected generally from time to time by the Asphalt Superintendent and Engineer of Bridges, each acting in his respective capacity, and is also inspected by the Chief Engineer.

Thus a serious error in design or construction, or the loss of funds through misappropriation, ignorance, or criminal waste, is effectively prevented.

Furthermore, nearly all the work is done under the eye and at the expense of the public, of whom a very considerable number are more

or less familiar with road building, and who, by appeal to the press, the Courts, or public officials, could cut short any gross incompetence, negligence, or dishonesty of employee or official. Mr. White

The Act provides that funds for the improvement and maintenance of all highways, other than Township Dirt Roads, shall be distributed as equably as possible throughout the sixty-six counties of the State, which effectively prevents the Commissioner from showing any undue partiality toward any particular portion of the State. Hence there is always one man who can be held responsible for results in carrying out the orders issued by higher authority, and, at the same time, in the decision of important questions, there is always one of competent knowledge to whom he can turn for advice.

All appointments to positions in the Department are made by the Commissioner, and are not subject to other approval, except in the case of a few of the higher positions, which are subject to the approval of the Governor.

It is important that the Commissioner be given much freedom in making these appointments, because, when men having considerable authority are scattered widely over the State, absolute loyalty to the "chief" is of the utmost importance, and can be obtained more effectively by personal appointment than by any system of civil service examination.

At the same time, as many of the positions require a high degree of executive ability, as well as technical skill and knowledge, it is important that the appointments be placed, as far as possible, beyond the reach of "political spoilsmen". For this purpose it has been decided that, in the future, as far as practicable, all the higher positions shall be filled by promotion, after careful examination into the candidate's technical knowledge and experience, as well as a careful inquiry into his previous record, in order to ascertain his executive ability, trustworthiness, resourcefulness, tact, and general ability, to get the best results out of all with whom he comes in contact, whether superiors, subordinates, equals, or strangers, these qualities, at least for the higher positions, being really more essential than a high degree of technical skill or knowledge.

It is needless to add that the Commissioner himself must be possessed of a high grade of technical skill, as well as great business and executive ability, in order to accomplish in the best manner the results expected of him by those to whom he is accountable.

William H. Connell, Assoc. M. Am. Soc. C. E. (by letter).—About two months ago a search for literature on "Municipal Highway Engineering Organizations" was made by the Library force of this Society. This search covered the past 5 years, and only one article dealing with this subject was found. Papers, however, have been written relating to the routine of existing highway organizations, and this has also Mr. Connell

been covered in some public works reports, including organization charts for highway departments, sometimes termed "street departments"; and it is apparent from some of these papers and reports that the officials in charge are making the best of the conditions under which they are operating, but that they are simply outlining and dealing with existing organizations, without comment or constructive criticism regarding the organization of a modern highway bureau. This being the case, it may be well to pause for a few moments and analyze some of the reasons for paying so little attention to the organization end of this important subject, which is second to no other municipal engineering division, from a business and engineering standpoint.

Highway engineering is not new, and considerable advance, resulting in improved pavements, has been made within the past 50 years, but this advance has been achieved by the efforts of a few engineers working under tremendous disadvantages, through lack of co-operation by the Engineering Profession and the fact that highway engineering has been considered seriously by engineers as a body only within the past few years. The highway work, in a large percentage of municipalities has been handled by all classes of officials from as many different walks of life (none of which gave them a claim to any qualification for the work), or by a city engineer who has busied himself with water, sewer, and bridge problems, the nature of which diverted his attention from highway work. The latter involves so much detail of an engineering nature that it is not apt to appeal to one who is wrapped up in the problems pertaining to other branches of the Profession. As a result, the highway work was allowed to drift along until the highway department was considered to be the property of the politician, and, still worse, only recently it has been used by some business administrations throughout the country to parcel out a few jobs to men probably deserving of some recognition; consequently, the highway bureau has been made to suffer through the appointment of men conspicuously unfit for the work. In this policy a great mistake has been made, as the highway bureau, in a large measure, is the principal show-case of the city government—the pavements representing the goods in the window—and it behooves any city or State administration, political or otherwise, to look to its highways, for the public is alive to the situation, and recent developments have proved that more people can be reached and satisfied through an engineering highway organization conducted on a high plane, than through any other branch of public work. An adequate organization, however, is essential, as a successful highway administration is dependent on conducting and controlling the work with the least friction. Ease of operation is the most important factor, and this can only be obtained through an organization commensurate with the demands on it.

The political view of the highway department was expressed only recently by a leading political light of one of the largest cities in America, who stated that the highway inspectors in his city were all right because "they used to be truck drivers and consequently knew the streets pretty well." This statement was made in all seriousness, and to an engineer. It is small wonder, then, that many highway organizations reflect little credit on our municipalities. In fact, it is remarkable that the few engineers who have devoted their lives to this great work should have succeeded in producing so many good pavements, for in these there has been very little improvement, as far as the standard types are concerned, within the past few years, or since engineers in general have become interested in the subject.

Much has been said concerning municipal and State highway organizations by new and inexperienced officials, who, instead of employing experienced engineers to plan their organizations, attempt to do so themselves, and often write articles on the subject which only too frequently are misleading, as they are not based on good engineering judgment coupled with experience in management on a large scale, both of which are necessary in planning such organizations or writing on the subject. Of course, a man familiar with business organizations for controlling work can plan a highway or any other kind of organization, but not without the advice of one familiar with all the functions and work coming under the jurisdiction of the respective organizations. Therefore, if it becomes necessary to plan a highway organization, it is far better to have the services of an engineer versed in the principles of organization and familiar with the work. In other words, the most economical, the quickest, and the surest means of attaining the end in such an undertaking is to pick a man whose qualifications come nearest to fitting him for the work, and, in the case of highway work, select a highway engineer.

With reference to the organization itself, the writer will assume that he is dealing with a large city, and that necessary modifications should be made, depending on its size. In a small city, of course, it would not be practicable to have as many separate departments as in a large one, but the principles set forth can be followed in the handling of the work. Instead of discussing existing organizations, an attempt will be made to outline a proposed highway organization commensurate with present-day requirements. In this it is most important to determine what functions should come properly under the jurisdiction of a modern highway bureau and just where this jurisdiction should start. The first step, or the laying out of the highways, should come under the head of "city planning," which is becoming more and more a special branch of the Engineering Profession, and the work is of such a character and the field is so broad that obviously all municipal engineering departments will necessarily have to be more or less

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affiliated with the city planning division, and the head of this division might be termed the city engineer. Therefore, it will be assumed that the highway bureau, after the layout has been planned, will start with the design of all work pertaining to the highways, including parkways, park drives, and small highway bridges. The larger bridges should come under a bridge department, as bridge engineering is distinctly a special branch of the Profession. After the design, of course, all necessary engineering work relating to lines, grades, and inspection, from the grading to the finished pavement, should be under the jurisdiction of the highway bureau. Before leaving the subject of design—in view of the tendency of late in our municipalities to create a central bureau of design—it might be well to call attention to the fact that the engineers in charge of construction and maintenance of highways would necessarily become specialists in this line of work, and should control and be held responsible for the design as well as the construction and maintenance. This is simply the logical procedure which should be adopted in any such work. If, therefore, there is to be a central bureau of design—which may be very desirable—it should be under the city planning division, and the designing—as far as the construction of highways is concerned—should be supervised by the highway bureau.

The next thing to be considered is one which is becoming more serious every year, namely, sub-surface structures and encroachments, which include all underground conduits, pipe lines, vaults, etc., steps, street signs, stands, and every kind of encroachment beyond the building line. These matters, together with permits and licenses for automobiles, vehicles, etc., should be under the supervision of a branch of the highway bureau. With respect to conduits, pipe lines, etc., the only solution of the problem for the protection of the pavement, convenience of the public, and general traffic conditions, would appear to be pipe galleries; but, in large cities, which are already paved and built up, this would require an enormous outlay. This question should be considered seriously, however, in growing municipalities.

Having provided for the design, sub-surface structures, and engineering work, including inspection, the next subject is the surface of the street, and here the first thing to determine is just where the jurisdiction of this bureau should begin and end. Generally, in all municipalities, the city controls the highways from house line to house line, regardless of the division of expense for the footways, paving, repaving, etc., which varies in different cities. This is a big problem in itself, and will not be discussed at this time. Assuming, therefore, that the city has such control over the highways, encroachments, street signs, license fees for automobiles, vehicles, etc., naturally come under the jurisdiction of this bureau, and right here is a big

problem—as is also the case with sub-surface structures particularly relating to vaults—that has not been solved, namely, an equitable and just rental value to be levied by the city for all kinds of projections—underground or above the surface—beyond the house line, and license fees on all classes of automobiles, vehicles, etc. This, however, as in the case of the division of expense for footways, paving, repaving, etc., is too big a question to consider at this time. To continue with the highway organization, it will be assumed that the supervision, control, and enforcement of all the regulations governing these matters come under the jurisdiction of the bureau of highways. Mr Conn

The next, and probably the most important, matter to be considered in connection with the work rightfully coming under the jurisdiction of a modern highway bureau is a branch of municipal work which is closely related to highways and plays an important part in their maintenance, namely the cleaning of streets. This branch of engineering work—for it is distinctly engineering work, as the life of the pavements and roads depends to a considerable extent on the kind of cleaning they receive—has never, except in a few municipalities, been placed on an engineering basis, and in many cities this work comes under the jurisdiction of a separate department, which not only results in increased supervision and overhead charges in connection with the care of the highways, but an overlapping of jurisdiction with regard to the control. In a well-organized highway bureau, including street cleaning, the street patrol inspection can be and is taken over by the men supervising the street cleaning. This makes it unnecessary to have a separate street patrol inspection force, such as is employed in many highway bureaus. The street cleaning forces are constantly at work, covering every street in the city at frequent intervals, and, being supplied with a memorandum book or blanks for the purpose, can write down observations of the conditions of the streets as well as supervise the street cleaning work.

The tendency of the day is to standardize and control work in a manner which will not only conserve energy and do away with unnecessary duplication, as far as individual effort is concerned, but will concentrate forces so that operations can be carried on efficiently and economically along the line of least resistance, as ease of operation is one of the most important factors to be considered in planning any working organization, and no step would accomplish more in this direction than the combination of the street cleaning and highway bureaus. This would have the added advantage of placing this work more generally on an engineering basis, which would undoubtedly lead, not only to more sanitary methods of cleaning, but to methods which would be more apt to take into consideration the maintenance and care of the pavements themselves than is likely to be the case where the street cleaning is not under engineering supervision.

In connection with the street cleaning work there is also the collection of ashes and rubbish and the collection and disposal of garbage. Although this work is not distinctly a highway matter, the collection and disposal of ashes, and rubbish particularly, is so much a factor of the problem of preventive street cleaning—in which the crusade against overloaded and leaky wagons, the throwing of papers, store sweepings, etc., into the streets, plays an important part—that it more properly belongs under the highway than any other municipal department. In a lesser degree, this is also the case with the collection and disposal of garbage. The writer, therefore, for the time being at least, or until a better solution of this problem is presented, will place both these matters under the jurisdiction of the highway bureau.

With reference to the division and subdivision of the work, assuming that all that comes under the jurisdiction of the bureau is distinctly highway work, the construction and maintenance and cleaning of streets should be under one chief engineer, with as many division and section engineers as the size of the city and the extent of the work requires. The clerical, including auditing, stenographic, and general office force, and the division of sub-surface structures and encroachments, including permits and licenses, should, through their respective heads, come under the chief engineer, as should also the division of contracts, specifications, and general office engineering. The work under these divisions should be delegated in a like manner, depending on the extent and nature of the undertakings over which the respective heads have control. Each of these subdivisions embraces so much detail, with the consequent opportunity for constructive criticism, that in the limited time assigned to this subject it would be folly to attempt to elaborate any one of them herein. Therefore only the main subdivisions and functions coming under the general principles set forth for a modern highway bureau have been taken up. The chart submitted herewith, Fig. 1, covers the organization outlined, and is intended for a city having a population of 1 500 000, and covering an area of about 130 sq. miles. The details will be left for future discussion, which must necessarily go on until modern highway engineering comes into its own.

Before closing, and in order that all may realize the magnitude of the work of highway bureaus in comparison with that of other engineering organizations, the writer calls attention, for example, to the extent of the undertaking of the Water Board of the City of New York in connection with the new water supply from the Catskill Mountains and vicinity. This work is said to be second only to the construction of the Panama Canal—the greatest piece of engineering work of the Twentieth Century. Now, no matter what may be the nature of the work requiring large expenditures, it is just as necessary to see that the money is spent judiciously and to the best advantage in one branch

Mr. Connell

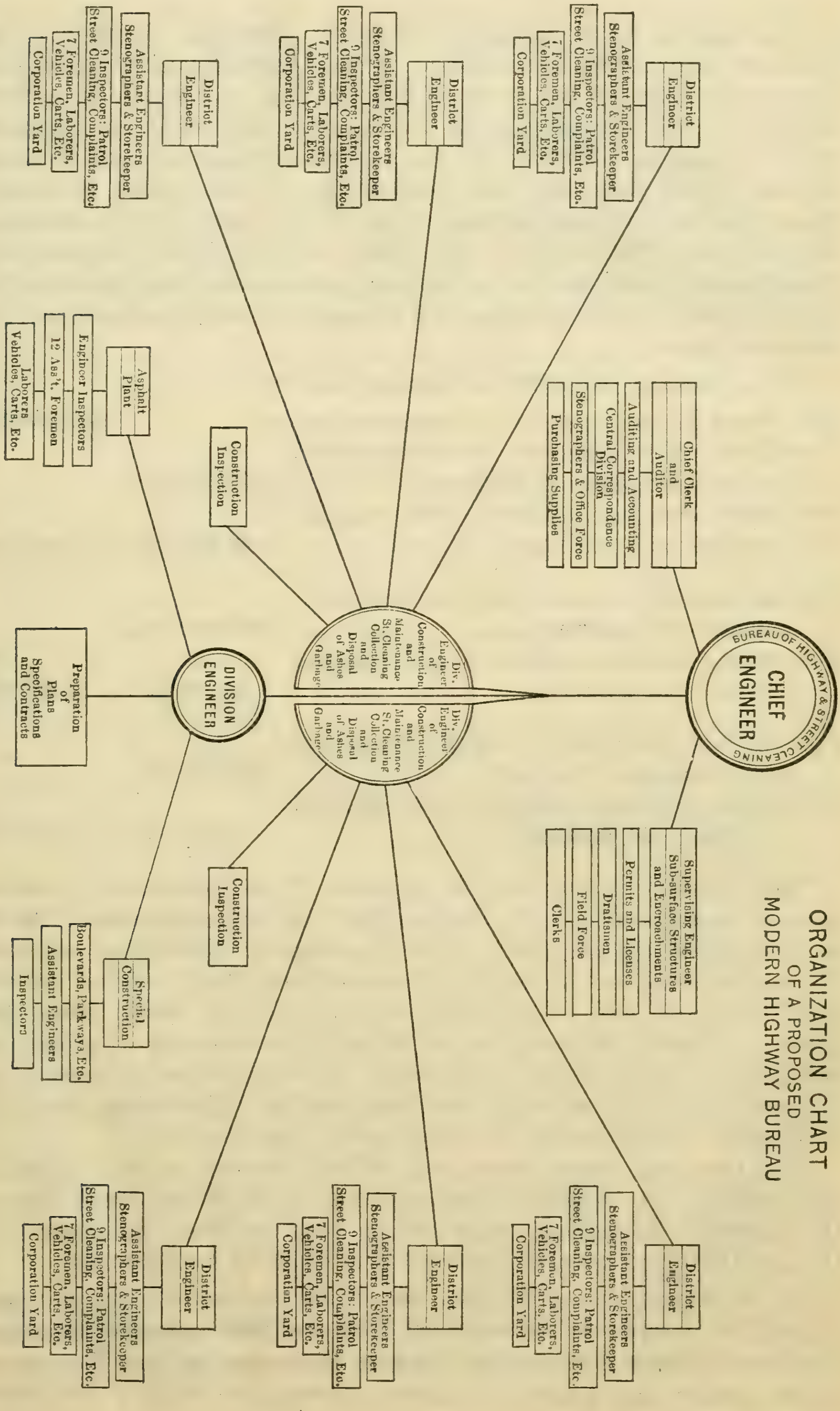

FIG. 1.

r. nell. of engineering as in another. Therefore, attention is called to the excellent engineering organization of the Water Board of New York City, with which all are more or less familiar, and which is, without doubt, the best organization of the kind in the world, as compared with the general character of the inadequate highway organizations having jurisdiction over the expenditure of large sums of money. Then compare the expenditures of the Water Board for construction for the years 1905 to 1913, with those of New York, Chicago, and Philadelphia, respectively, for highway construction and maintenance, street cleaning, and collection and disposal of ashes and garbage, for the same period:

From January 1st, 1905, to January 1st, 1913:

The Water Board expended on construction work........		\$87 550 540
New York City expended for construction and maintenance of highways.........	\$108 523 030	
For street cleaning, collection and disposal of ashes and garbage.................	66 953 463	175 476 493
Chicago expended for construction and maintenance of highways..................	\$36 846 751	
For street cleaning, collection and disposal of ashes and garbage.................	16 209 179	53 055 930
Philadelphia expended for construction and maintenance of highways..............	\$17 445 085	
For street cleaning, collection and disposal of ashes and garbage.................	12 904 030	30 349 115

Now, without going into the question of the personnel of the respective organizations, it is obvious that the same importance should be attached to the character of the organization controlling one class of work as another, per dollar of expenditure; but, judging from a purely business and engineering point of view, it would seem that there is a general tendency not to regard it as necessary to provide as adequate an organization to supervise the expenditure of moneys on highways as is the case with other engineering undertakings, and this is a matter which should be given serious consideration.

The foregoing figures come from reliable sources and are substantially correct. The details of the personnel of the Water Board and of the highway engineering organizations have not been stated as it is thought that their general character is sufficiently well known.

r. es. E. W. James, Assoc. M. Am. Soc. C. E. (by letter).—It is almost impossible to separate the technical from the administrative organization for highway construction, because highways are not like waterworks, sewers, gas plants, or electric service, but hold a distinctive

place among public works. The highway is the first public work undertaken by government, and its construction and upkeep is both a duty and a right of the community.* Mr. James.

In many States the road taxes are still paid in labor. This, which of course is a poll tax—is always a road tax. Wherever local enlightenment has become sufficient to provide a money commutation for the labor, the proceeds are returnable to the district from which they have been collected for use exclusively on the roads. The origination of the road tax in the form of labor was in part the result of following the English system of local government, which alone was generally understood by a majority of the early settlers in America, and in part the result of the poverty in money resources and the richness in brawn and time resources which characterized newly settled communities.

It is rather strange that the labor tax has persisted in some of the States, but wherever it still exists, there is usually a local reason which is considered sufficient to explain adherence to so antiquated and inefficient a method. That the old law stands is no doubt due somewhat to the fact that though conditions have changed, the administration of the law has continued in the hands of unskilled men who have been able to see neither its defects nor any way to remedy them.† Its persistence, however, has caused one very pronounced condition of mind among the citizens of a large part of the country. Most men's grandfathers who lived west of the Appalachians built their own roads; later, their fathers built their own roads; and, consequently, they of to-day believe that they can build their own roads, and they leave the road labor tax on the statute books year after year.

This idea is much more prevalent than the casual observer will usually admit, but when we see that communities, in which the traffic has grown to such proportions and weight as to demand a much better type of construction than formerly, continue to depend on the labor tax and try to build roads without proper engineering, without proper financing, without even proper tools in some cases, we catch a glimpse of the inertia which characterizes a persistent idea. When the rural county citizen undertakes road work, he continues to believe that what his grandfather could do, he can do, and he, more than likely, dispenses with engineering advice, if the local good roads association does not force him to seek it.

This attitude of mind was embodied in the road laws of practically all the Southern and Western States until within the past 5 or 10 years, and is still reflected in many of them. The following States continue to collect all or part of the road tax in labor: Alabama,

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*In elaboration of this idea, Dr. D. F. Wilcox, in "The American City," makes some interesting comments.

† "Road Legislation for the American State," by J. W. Jenks, Am. Econ. Assoc., Vol. IV, No. 3, May, 1889, p. 23.

Tennessee, North Carolina, South Carolina, Georgia, Florida, Mississippi, Texas, Louisiana, North Dakota, New Mexico, Oklahoma, Arkansas, Montana, Kentucky, South Dakota, Colorado, Idaho, Illinois, Indiana, Minnesota, Nevada, Missouri, Pennsylvania, and Wisconsin.

Almost entire lack of engineering organization in connection with highway building in a large part of the United States has been the direct result of these conditions. Further, whatever efforts have been successful during the past few years in putting new statutes on the State books, none has succeeded in eliminating entirely the common idea that the farmer is as good a road builder as modern days can furnish.

The county and town system of road administration which obtains almost everywhere, places the construction and maintenance of roads in the hands of the county commissioners. In many cases, the commissioners have authority to divide the county into districts and appoint a supervisor over each district. Frequently, a penalty is prescribed for a man who, if appointed, refuses to act as supervisor. These men are furnished with a list of taxable polls in their several districts, and their duties include the collection of the road tax. They summon the road hands under the labor tax system, on the conditions fixed by law, and see that the legal time, or so much of it as may be required, is spent in working the roads. Those road hands who choose are usually permitted to pay out at a rate varying from 50 cents to $3 a day.* The supervisor collects the commutations, gives a receipt, and spends the proceeds on the roads in his district. His accounts usually stand like this:

Collected in road taxes....................	$787.50
Paid out for road work....................	787.50

Generally, a large part of the sum charged to road work goes, by tacit consent, as his perquisite, into the supervisor's pocket.

There has been a somewhat strong movement away from such conditions during the last 10 years, and we find many new laws drawn to relieve us of such ridiculous and wasteful methods of road administration. Still, the remedy has not been general, by any means, and there are some interesting clauses in the new laws. In the Alabama Road Law of April, 1911, for instance, the State Highway Engineer "may * * * be consulted by the county commissioners," and in cases where projects involving State-aid funds cost more than $3 000, "the highway engineer, with the consent and advice of the proper authorities in the county, may prepare plans and specifications" for executing the work by contract.

In the Arizona law of June, 1912, it is declared that "all roads and bridges, when constructed, shall thereafter be maintained and im-

* The common rate is $1 per day, but it ranges from 50 cents in South Carolina, to $3 in Nevada.

proved when necessary, * * * under the joint auspices and direction of the State Engineer and Board of Supervisors, * * *." Mr. James

The Virginia laws of 1904, 1906, 1908, and 1910, are a respectable effort to do better things, but, from the limitations placed on the constituted road officials, it would seem that the General Assembly believed that the county commissioners knew more about road engineering than the State engineers whom they might employ under the law. Iowa has recently passed a new road law (April, 1913) which is a departure in legislation. It is the most systematic effort to correlate road work done at county expense that has yet been made. It could well be studied by the legislatures of three-fourths of the States. The new State of New Mexico goes one step farther in its road law of 1912. A State commission is created, and one of its duties is to appoint all county road commissions, each of which is composed of three members.

In general, however, except where State funds are involved, there is no engineering supervision required over county road work in the rural sections of most of our States. The counties thus conditioned spend, according to the most accurate reports attainable, about $61 000 000 annually on their roads. This is 40% of the annual road revenues of all counties.*

As soon as a State organizes a highway department and sets aside funds to aid the counties, the first systematic engineering organization is developed. The types of organization are simple and rather uniform for most States. They are confined to two common systems: one has a corps of assistant engineers reporting directly to the chief engineer; the other has a group of division engineers reporting directly to the chief. In a few cases, notably in Ohio and New York, there are deputies having general charge of a department of the work, as construction, bridges, maintenance. In these cases division engineers report to their respective deputies.

Under the division engineers there are resident engineers and inspectors assigned to projects.

A review of these actual conditions presents several questions well worth discussing. It is obvious that much of the reluctance on the part of poor counties to seek engineering advice is because there is no man available in the county, and an outsider—besides being an outsider, which frequently is considered more or less objectionable in itself—cannot be secured at a compensation warranted by the county revenues. As we see, therefore, it is not until the State steps in and gives the county funds that there is any engineering supervision of a large part of the county work. Cannot some simple engineering organization be created to deal with this situation?

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*Cf. Jenks, *supra*, p. 43. The principal exceptions are Massachusetts, New York, New Mexico, and Iowa.

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At a hearing of a legislative committee in a Southern State, the writer had an opportunity, about 2 years ago, to suggest a plan that he still believes has merit. The plan comprises a highway commissioner, paid by the State, drawn, if possible, from the staff of the State university or mechanical college, so that the additional salary would be small; a corps of division engineers assigned to groups made up of from eight to ten counties.

The commissioner would have general supervision of the division engineers, and they would have charge of all road work in the counties of their divisions. Many of these counties never would undertake macadam construction. Their roads, because of lack of local funds, must remain earth, sand-clay, or gravel for many years to come. The duties of the division engineers would be to supervise the expenditure of the county funds. The commonest needs in most rural counties are: the substitution of some material besides short-leaf pine for box culverts, the construction of adequate ditches, the proper crowning of the traveled way, direct and regular alignment, the intelligent mixing of sand and clay, the drainage of spring holes and swampy places, and the removal of vegetation from old mudholes. The division engineer's duty would be to see that suitable construction gangs are organized to work over the county and do their work properly. Labor taxes should be commuted by cash payments, and the money used to pay the road gangs. The old supervisors should be discontinued, and a group of foremen substituted, who, if possible, should be retained permanently, and trained in their work. When bridges or culverts are needed, the division engineer or commissioner can furnish plans, and the division man should see that the work is done in a correct and efficient way.*

This plan would much reduce the cost of engineering to the counties by a general sharing of expenses, and would be productive of good results in several directions. It would do away with the supervisor system, and would tend to remove the work from political influences. It would concentrate expenditures, and prevent the general custom under the ordinary county method of working every road in sight to please as many voters as possible, with the obvious result that the work is spread out so thin that the rainy or winter season destroys it all. It would create some standardization and begin the elimination of hopelessly ineffective methods used year in and year out by the county officials without apparent betterment of the roads. It would create in time some sentiment favorable to the employment of private engineers and a general raising of the standard of all public works. This latter circumstance would be a most happy result, for the need of adequate engineering in small municipalities through-

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* Jenks, *supra*, p. 48.

out the South and West, and indeed almost everywhere in the country, is glaringly apparent on the most casual investigation. Mr. James

When State funds are appropriated, conditions are at once produced that direct attention to another question. Shall the organization distinguish between construction and maintenace? The last report made by the New York State Commission recommended increasing the number of divisions and combining the responsibility for maintenance and construction. This recommendation was interesting and significant, coming from an organization which has had such a varied history as the New York State Commission. The State Legislature, acting in part on this recommendation, passed a law permitting an increase from six to not more than nine divisions, and otherwise embodying the recommendations. The type of commission was also changed, and a single responsible head provided. The present highway department in New York works under this new statute.

The labor organization and type of inspection required on construction, on one hand, and on maintenance, on the other, are so widely diverse that it appears to the writer that some separation of the two classes of work can be advantageously made. In the first place, if the division or district heads do not have to split their duties between the two classes of work, each can cover a larger territory. Further, if there is a system of standards used, the maintenance divisions can be enlarged beyond the size of those possible for construction divisions. This is so because, in any system so large as to demand division or district organization, the quantity of work makes construction by contract practically compulsory, and contract work requires constant and detailed inspection. On the other hand, maintenance by either the patrol or gang systems is force account or administration work, and as the patrolmen, foremen, and gangs are trained, they require less and less minute supervision. As a further consideration, it is not a bad plan by any means to establish as soon as possible the separate identity of the maintenance organization; because at last this will be the only one left, as in the case of the French *Routes Nationales* today. Trained engineers are generally required to take residences, whereas a division, when entirely under maintenance organization, would probably require no such resident engineers. In very large maintenance divisions assistants might be necessary to handle the larger re-surfacing projects, but in most cases trained foremen would be able to do a large part of the work under general orders.

On the other hand, an indisputable advantage comes from the vertical organization of divisions. If an engineer has entire charge of a restricted mileage during construction, he is doubtless the best prepared man obtainable to handle the maintenance of the same sections. He has watched the construction and knows the weak spots, the bad soil, the softer stone, and the doubtful drainage arrangements.

r. nes. He knows the county thoroughly, and should have developed amiable relations with local officials and the citizens along the roads. All these matters would count much in a man's equipment for division work, but there is no insurmountable reason for a division man not leaving record maps sufficiently complete to inform a successor, instructed to take over roads for maintenance, in all the major and many of the minor conditions along the sections under observation. This matter of small-scale record maps is capable of large and valuable development. Further, a man detailed to maintenance, and concentrating his attention on it, should soon acquaint himself with the conditions in his division. It is seldom that labor used in construction stays to maintain, and, therefore, no especial advantages in the line of securing skilled laborers can be expected under either system. A new labor and supervisory organization will generally have to be created, and, in most instances, be recruited from the local supply.

Another matter that, from the standpoint of efficiency and economy, is of especial importance, is the place occupied under the State organization by the county road administration. Three States, New York, Iowa, and New Mexico, have statutes specifically placing the county road engineers or supervisors in the State system. There is no denying that this innovation (for, in the present condition of our State road laws, it can certainly be considered such) has little about it to be unfavorably criticized. Of course, when within a State the question of local autonomy comes up in any form, we always find some who oppose any restriction on home rule; but, if we analyze the basis of home rule, we find it chiefly concerned with matters of conduct. Now, road building is not a matter of conduct. Our fellow-citizen does not understand why his road money should not be economically expended by competent engineers. Now that two or three States have led the way, it may be hopefully expected that there will be a further move in the direction of placing some check on the indiscriminate expenditure of county funds under a local system which is worse than simply inefficient. This can be done, even when no State-aid feature enters in as a sop to the county. When such allotment does appear, there is no reason why supervision should not extend beyond the limits of the State-aid roads.

In Iowa the county engineers and the county officials, as far as road matters are concerned, are under the supervision of the State commission. In New Mexico the county road commissions are appointed by the State commissioner. In New York, county and town supervisors are under the division engineers, and in Illinois, the county engineers are appointed from a list of eligibles satisfactory to the State commission.

Just how far the State organization shall be indicated by law is a matter of debate. The writer believes that few States err on this point

at the present time. Generally, except in three or four cases, the detail of the highway department organization is not carried beyond the State highway engineer under various titles. The details of organization are left to the administrative heads and the engineer. This is as it should be, until we find some organization that has details of particular merit capable of wide application. This freedom, however, does not preclude placing the county road authorities by law under the supervision of the State organization. Mr. Jame

Many of the State departments to-day are advisory or nominally supervisory in their duties, and it is this restriction which, in spite of freedom in departmental organization, restrains their usefulness in so many cases to almost negligible limits. Where the great gain in efficient expenditure must come is in the county work, and the State engineer is helpless, however competent he may be, unless he is given some element of authority over the local activities. Merely advisory work is very often gratuitous. Some county officials consider it almost meddlesome. It is these local Solons that good organization must circumvent.

A review of the several State laws shows that the highway departments suffer from one very general condition. There are few single responsible heads. We are familiar enough with actual working conditions in some States to know that the work suffers, or has suffered, seriously because responsibility is not concentrated in one administrative official. It is very common to have the law stipulate that the engineer shall work in conjunction with an administrative official, sometimes even with an administrative body, of equal or greater authority. Arizona presents a case of this sort. In some States work is being done successfully, because, with a poorly devised law, a commission of three or five members has turned the work over to one of their number, or to a chief engineer, who acts practically as a single head. This has been conspicuously true in Virginia. The ill success of a many-headed commission can be seen in Rhode Island. There a commission of five members, one from each county, has had active charge of work, and instances have occurred where commissioners, visiting construction in progress, have changed the orders of the engineer in charge.

With regard to details of organization under the commission or responsible heads, the laws in most cases leave the highway department free to adopt such system as seems best fitted to accomplish the ends and purposes of the statutes. Were there a general system of single responsible heads, many States under their present laws, could develop thoroughly efficient organizations. The responsibility for the organization would lie with the head, and he would generally be unrestricted. There are thirty-eight States which have some form of highway department, but it is doubtful if more than fifteen of them

es. can develop efficient highway organizations, because of the divided authority and responsibility residing in several heads.

ett. CHARLES J. BENNETT, M. AM. SOC. C. E.—With one or two exceptions, the speaker is impressed with the general soundness of the arguments presented by Mr. Connell. In the first place the formation of a standard organization for a highway department in a city is not entirely feasible. There are so many difficulties in the way of standardizing any particular department of a city that it would be impractical to consider any more than general principles to be used in all cities, instead of trying to standardize a system for one department in all.

For instance, a standard system applicable to New York City, Chicago, or Boston would certainly not apply to a smaller place, on account, not only of the size of the city itself, but the general geography, characteristics of the people, and the system of government. It seems plain, therefore, that if a standard system is to be devised, all cities in the United States should be classified and a system of organization for all departments formed, so that the entire operation of city government could be along standard lines for different types of cities.

This seems to be an ideal which cannot be reached, and the problem, therefore, must be to apply general principles to engineering organizations which shall tend to improve conditions which are bad at present. These applications should also furnish engineers with information which can be classified by engineering societies, so that costs and systems may be contrasted and certain valuable improvements in results gained.

There is no question of the need of an engineer to supervise not only highway departments, but all public works of any character. Therefore, a Municipal Highway Engineering Bureau should be a part, and a very large part, of an Engineering Bureau of Public Works, and public works of all kinds should be under the supervision of a trained engineer.

This principle, if applied to cities, will work with incalculable benefit, both in the improvement of the construction of public works and in their economical operation after completion. Granting this, therefore, Mr. Connell's statement as to the need of an engineer to supervise all highway work is evident, and the supervision of this work, as he states, should include, not only the actual construction and operation of these public works, but also the design of the different structures as well as the outline of the general plan to be carried out in the expansion of the city.

Now, a word as to the qualifications of an engineer in public service, and the difficulties of securing one who is competent to follow up work of this class: Engineers by training should be fitted to do all things necessary for the operation of such a magnificent department

as has been conceived in control of the public work of a city or State. That means that the engineer must be not only a technical man but a business man who can see both the scientific and financial sides of the problem, and determine the advisability of spending any given sum of money for a certain purpose. Mr. Benne

The second qualification means that the engineer must take a broader view of public work than he generally does.

In considering the operation of any city or State engineering department (and the ideas here outlined apply, it seems, particularly to the highway department), there is need of a better training for engineers in keeping accounts and supervising routine office work. The success of any highway bureau, or any other bureau which is under the supervision of an engineer, depends on its accounting system as much as on any part of the whole situation. The expenditures must be kept within appropriations, and work must be planned and executed in such a manner as to gain the full benefit, so far as possible, from the expenditure of moneys allowed. Therefore, the engineer should be trained, and highly trained, along business as well as technical lines. The natural tendency would be for an engineer to be too much given to minutiæ and complicated methods. The endeavor should be to devise a system of accounts as simple and free from unnecessary complications as possible.

This discussion has followed general lines more than the particular ones specified in the topic, but these ideas—which must be considered as general, rather than specific—can be applied to a highway engineering organization.

In connection with the discussion on highway work, both municipal and State, it is interesting to note the diversity of opinion among engineers as to the value of the work done. Within the past year a prominent engineer has told the writer that there are very few engineering problems to be faced in highway work; that this work is of the simplest nature, and, by inference, almost any one could become a highway engineer.

To the ordinary man's mind, the highest type of engineering would be that which would do, in the best manner, both the simple and the complicated things.

There are comparatively few very large structures or projects to be carried out by engineers, but, as indicated by Mr. Connell, the largest amount of money spent in engineering work is for the operation of municipal and State highway departments, and the results of this work are open to critical observation by all who are in possession of five senses.

Although the engineering problems may be simple, the problem of spending public money in an economical manner, is, by far, the hardest one which the engineer has to solve, and if this can be solved

by improved business methods, as well as improved technical methods, it seems that we will have done some of the highest class of engineering work.

In conclusion, the speaker endorses Mr. Connell's statement, that the highway or other departments of cities or States should be freed from political influence, and that the spoils system should be entirely eliminated in the operation of these departments. Only a moderate amount of success can be attained when any system of politics, whether it be good or bad, is allowed to interfere with a municipal or State department, and when these departments are entirely free from political interference, then, and then only, will ultimate success be gained.

A. W. Dean, M. Am. Soc. C. E.—Very recently the word "efficiency" has become a particular favorite, not only with those who are directly concerned with State and municipal affairs, but with all who are interested in the administration and execution of public works. Witness the large number of governing bodies which have created commissions or boards of efficiency and economy to analyze and criticize the acts of public officials, and, by their advice, minimize waste and extravagance in the expenditure of public funds.

No department in State or municipality lends a greater field of action to such a board than the highway department, when competency is not the prime requisite in the selection of heads and assistants; and, on the other hand, no highway department consisting of competent men selected for their particular fitness for the positions they occupy will furnish a fair-minded efficiency board with any cause for reprimand or public criticism.

The preceding statements are made only to emphasize the fact that no highway organization can be successful unless competency is the first and only consideration in the selection, not only of the head, but of the entire body of the organization. Competency and efficiency are inseparable terms, for without one the other cannot exist.

A detailed description, either in words or graphically, of an organization adaptive to any and all conditions is obviously impossible. Highway laws in the several States vary widely, as do also conditions to be met in municipalities of varying wealth and population.

Supported by laws prescribing a well-defined policy in the expenditure of State funds, a single responsible head is preferable to a body of three or more; and, on the other hand, if the State has not, by legislation, fixed a general policy, a body of three may rightly be considered preferable. With or without a prescribed policy, a single responsible head with an advisory council having limited powers should prove efficient in carrying out large or small undertakings.

In a municipality it has been well demonstrated that a single responsible head is sufficient.

No organization can be long successful unless it is arranged below the head in military order. If the organization is small, the head may act as the chief executive; if large, there must be one chief executive charged with carrying out the general policies and orders of the head, as a division of responsibility at this point tends toward duplication, uncertainty, and consequent inefficiency. Mr. Dean.

Divisions and sub-divisions below the chief executive officer are dependent on the scope of the work to be done; in State highway organizations a suitable number of division engineers are charged with responsibility for both construction and maintenance in their several divisions. In municipal organizations each division should include the entire municipality, with a division engineer in charge of each branch of the work, that is, one in charge of bridges, one for street construction, one for maintenance, etc. For large undertakings the division may be subdivided into districts, with district engineers in charge responsible to the division engineers.

To summarize and further present the matter briefly, the following outline of highway organization is submitted:

1.—A single commissioner, or three commissioners, as quite definitely recommended above;
2.—A single responsible chief executive officer;
3.—As many divisions as may be necessary, each under a division engineer;
4.—Sub-divisions or districts, each in charge of a district engineer;
5.—Minor assistants.

All appointments should be governed entirely by fitness; there should be no changes in the personnel, except for proper cause; there must be no division of responsibility; all orders must be transmitted as in military practice.

Henry W. Durham, M. Am. Soc. C. E.—It is of interest to have confirmed by Mr. Connell's search, the fact, apparent to all who have been charged with city street maintenance and repairs, that there exists little constructive criticism based on broad lines as to the methods of carrying on such work. Destructive criticism abounds. Mr. Durha

Reports of citizens' committees denouncing indiscriminately pavements of bygone days, or the non-use of some special, possibly patented, cure-all street surface, the promoter of which has had an accelerator at work, illustrated by views of the depression where the prominent citizen's automobile was bumped, and accompanied by half-baked legal recommendations based on the American citizen's ineradicable impression that the millennium can be attained by "passing a law"—these, and the annual reports telling what has been done with the tools we have at hand, and the textbooks that tell much about good

r. pavements, but little as to paving organizations, comprise most of our
iam. highway literature.

There have been, it is true, studies made and plans advocated by efficiency experts for the better handling of the routine work of existing highway departments, or for their improvement in details, but the results of Mr. Connell's investigations would seem to indicate that little has been written on the fundamentals of organization and the proper scope required of the department in charge of highways in a great city. This is possibly due to the fact that in America the demand for improved street surfaces, caused by the increase in number, weight, and speed of vehicles using the highways, is coincident with the arrival of conditions causing their upheaval to a maximum degree. In no other part of the world at the present time, and in no previous period of the history of the United States, has there been anything paralleling the amount of rebuilding and shifting of what were supposed to be established trade and social centers, that is to-day going on in all large municipalities.

While the general public is demanding the construction of streets with absolutely smooth surfaces for availability for motor race-tracks, that shall be unyielding and without surface wear, so as to endure for all time, constructed over subways and pipe galleries to avoid all future surface disturbances, it is at the same time abandoning whole districts devoted to residences and light traffic, and filling them with tall office buildings which, when the expected office occupants fail to materialize, are turned into factories having a population in some sections of New York larger than the great mill towns of New England; and is demanding that these buildings have adequate service for telephone, electric light and power, and water supply, including high pressure for fire protection; and, finally is planning new lines of subways to carry away the surplus population brought to these districts, and to open up new possibilities of change. To harmonize these conflicting demands would appear to be an impossibility. It is under such conditions that officials charged with the maintenance of municipal highways are to-day carrying on their tasks.

To construct and maintain pavements under present-day traffic is an occupation in itself. When to that is added the fact that in no great city can the highway officials determine with certainty whether the pavement laid to-day will be suitable for the traffic to be found using it 5 years hence, it will be seen that they are entitled to the benefit of the best constructive criticism available, and that the fault-finder, who devotes what knowledge he may possess to enlarging his own newspaper reputation by the cheap method of criticizing obvious defects, is actually blocking progress.

Mr. Connell has taken up the problem in the right way, and has indicated an outline for the commencement of efficient highway or-

ganization. It may be of interest to have the subject also treated from other standpoints. In the course of an investigation of municipal highway work in European cities, made during the past year, some points were noted as to their methods of solving this problem. Mr. Durham.

Two general methods are followed in different parts of Europe:

1.—A strongly centralized authority under which the entire city is sub-divided into a number of smaller unit organizations, each within its district having charge of all municipal work, closely governed by the central office;

2.—Local control where, in the city as a whole or as sub-divided in each of a number of independent units, a single head has control of all city work, with sub-divisions, each separated and independently charged with a single class of work.

The first system is best exemplified in Paris. In the organization of the street service and for all construction work in the highways, it is divided into seven sections, each headed by a resident engineer who has control, not only of paving work but of the laying of water pipes, sewers, and the supervision of any sub-surface work of private corporations. At the head of the bureau of street service is an Inspector General with necessary deputies and office organization, having under him several chief engineers, one in charge of public streets and street lights to whom the resident engineers primarily report; another at the head of the work of street cleaning; one in charge of gas and electric light mains; the fourth in charge of water and sewers.

Each chief engineer has the necessary office force, and several resident engineers are under his orders for special work and for the management and maintenance of work outside the city; but, within the city, all construction work on the streets is under the direction of the street service engineer in charge of each district. Each in his own section takes care of the paving work; lays water pipes; builds sewers; and supervises duly authorized construction on the part of gas and electric companies, so that no pavement can be torn up except by his orders or with his permission.

In London, on the other hand, the control is almost entirely decentralized. There each of the thirty different boroughs has its own city engineer or surveyor responsible only to the Borough Council for orders and directly in charge of all street paving, repairs, cleaning, and lighting. The works of general utility to the entire city, such as water supply, main-drainage sewers, and municipal tramways, are controlled by special boards independently of the boroughs, and do any necessary street work under the supervision of the local borough authorities. Smaller English cities have usually a city engineer, charged with control over all the city works, having under him a Department of Highways headed by a road surveyor, and an organization

Mr. Durham. of assistants whose duties are confined to the maintenance, repair, and cleaning of highways. With some modifications, the system in the German cities is very similar.

In the first of the two systems there is one responsible head over all classes of work carried on in each unit—the unit being some convenient sub-division of the entire city—and though the different divisions of city work are each under special organizations, the actual execution, in theory at least, is controlled so as to render lack of co-operation or interference of different classes of work impossible.

In the second system, in the city as a whole, or in each of its independent units, in the case of large cities, there is centralized control to the extent that all construction, with the exception of water-works, is under the direction of a city engineer, but the special varieties of street work are usually carried on by independent sub-units. Of course, many exceptions are found in any necessarily brief and general classification like this. For thoroughness and theoretically perfect, systematic, and orderly procedure, the French method of organization seems to leave nothing to be desired.

The city work of London, on the contrary, is carried on under what is probably the most illogical and unsystematic arrangement that could be devised, but, judged by practical results, London presents probably the best appearance of any city in the world, as far as the streets are concerned, and Paris is constantly in a torn up and poorly maintained condition.

In Berlin, a city about equal in size to Paris, but divided into a number of independent city or borough governments, without a central control, but very thoroughly organized as units, the results show good conditions in some districts and poor in others. The practical deduction to be drawn from the comparison of these methods, not on paper but as worked out in the three great European cities, is that no system will produce results automatically, and that good results can be produced under adverse methods by a people accustomed to orderly procedure.

It will be seen that the scope of a department in charge of public highways can be very broad or very much restricted and still be in accordance with good practice in some prominent city.

Mr. Connell's plan seems to follow the French system, but without going far enough for completeness. If the work of the street department includes cleaning, it may well go farther and take in sewer, lighting, water and other city works requiring the use of the streets. If we believe, like Sterne, that "They order this matter better in France," we may well consider that a model city works department should be based on that of Paris, which, after co-ordinating all possible interests, furnishes ample opportunity for the card-index and loose-leaf expert to devise interlocking systems to his heart's content

until he arrives at his Utopia, where the Inspector Generalissimo, by a system of push-buttons at his desk, can run his department and never see the work. Mr. Durham

The weak points which make this system difficult of application lie in the question of personnel. We have in the United States no trained class of public works officials, no body of Government engineers organized like the French Corps of Bridges and Highways, and we have frequent elections and still believe at heart that "to the victor belong the spoils" including the right of spoiling the work done for us by the vanquished. It is hard enough to find one civil service man to fill a responsible position; to find seven equal and uniform would be almost impossible. In the light of possible conditions, it would seem to be a better policy to concentrate along all classes of work, organizing the Bureau of Highways into sections for doing specific things in charge of the men best qualified to do them. An outline of such an organization, based on experience in Manhattan Borough, but making improvements in some existing conditions, is somewhat as follows:

The head of the Bureau of Highways has charge of all construction and maintenance work on street surfaces between building lines, and issues and controls permits for necessary street openings of whatever nature, under the direction of a Commissioner of Public Works whose authority includes, besides highway construction, the sewers and other sub-divisions and in whose office all financial matters, including auditing, contracts, etc., are taken care of.

To avoid duplication of work, a single Bureau of Design and Survey carries on all necessary work for the different construction bureaus in the department, its work for each, however, being under the direction of the head of that bureau. This applies, not only to the making of preliminary and contract plans, but to the furnishing of parties for surveys, giving grades, and making measurements for final estimates.

Under a Chief Clerk is the control of all correspondence and records of the bureau, including the necessary organization for filing, recording, and handling data for the maintenance of a live record as to street conditions, necessary repairs, etc.

As has been pointed out, the control of street openings is an essential preliminary to the work of any highway department. No system, however complete, will be of more value than as a historical record unless some plan lies back of the causes necessitating such openings. It will be essential ultimately that our great cities realize the desirability of the adoption of a systematic plan for sub-surface structures for new districts and take up the problem of an orderly re-arrangement of these structures in the most congested sections. Thus far, charter rights of individual companies and the unwilling-

Mr. ·ham. ness of various departments to co-operate with one another have rendered futile any attempts in this direction, resulting in a tangled mass of sub-surface structures in our central districts, with locations frequently unrecorded, and the repair or extension of which requires an undue amount of street surface disturbance. Pipe galleries under all streets are a dream of the idealist, but an orderly planning which shall place under footways the lighter house-service lines, leaving the large mains under the roadway at regulated intervals, and shall do away with unnecessary duplication of facilities in any one street, and a limited provision for pipe galleries in the most crowded sections, is not impossible of attainment. It will require, undoubtedly, amendments to the charter, both of municipalities and corporations, and the vesting of the right to order changes in the body controlling the city policy. With such a condition existing, the permit office of the Bureau of Highways will have actual and not merely (as at present) nominal control over all street openings. This can be done without interfering with the rights of any other department or corporation to get at their structures for emergency repairs. The usual procedure will involve application to the permit office in advance of making any opening in the pavement, and a deposit of sufficient money to cover the cost of restoration, inspection of sub-surface work, back-filling, and subsequent repair and maintenance.

After street openings are properly controlled, the making of repairs necessitated by ordinary wear is of next importance. For convenience, this work has been placed in two divisions: one having charge of all repairs to block pavement, including stone block, wood and asphalt blocks, a large part of which work can be done by gangs with a limited quantity of new material, and repairs to sheet-asphalt pavements, which require the use of an asphalt plant, either owned by the city or operated by contractors. To lay out the work of these divisions properly, a system of street inspection is essential, and this is done by a division of patrol inspectors, each having a definite district and making daily reports of all defects, which are immediately plotted on an office map from which the day's work for the gangs can be laid out to the best advantage. Another division looks after all matters pertaining to sidewalks, whether carried on by the city or the property owners, and has control of such other matters as street signs, removal of incumbrances, etc.

Finally, as the perfect non-wearing pavement has not, as yet, been discovered, it becomes necessary to provide for renewals of street surfaces from time to time. This, for the present at least, is best done by contracts, under the supervision of a division engineer with a necessary organization of assistant engineers and inspectors.

Whether the street work is performed by city forces or by contractors, the purchase of new materials and their conformity to specifi-

Mr. Durham.

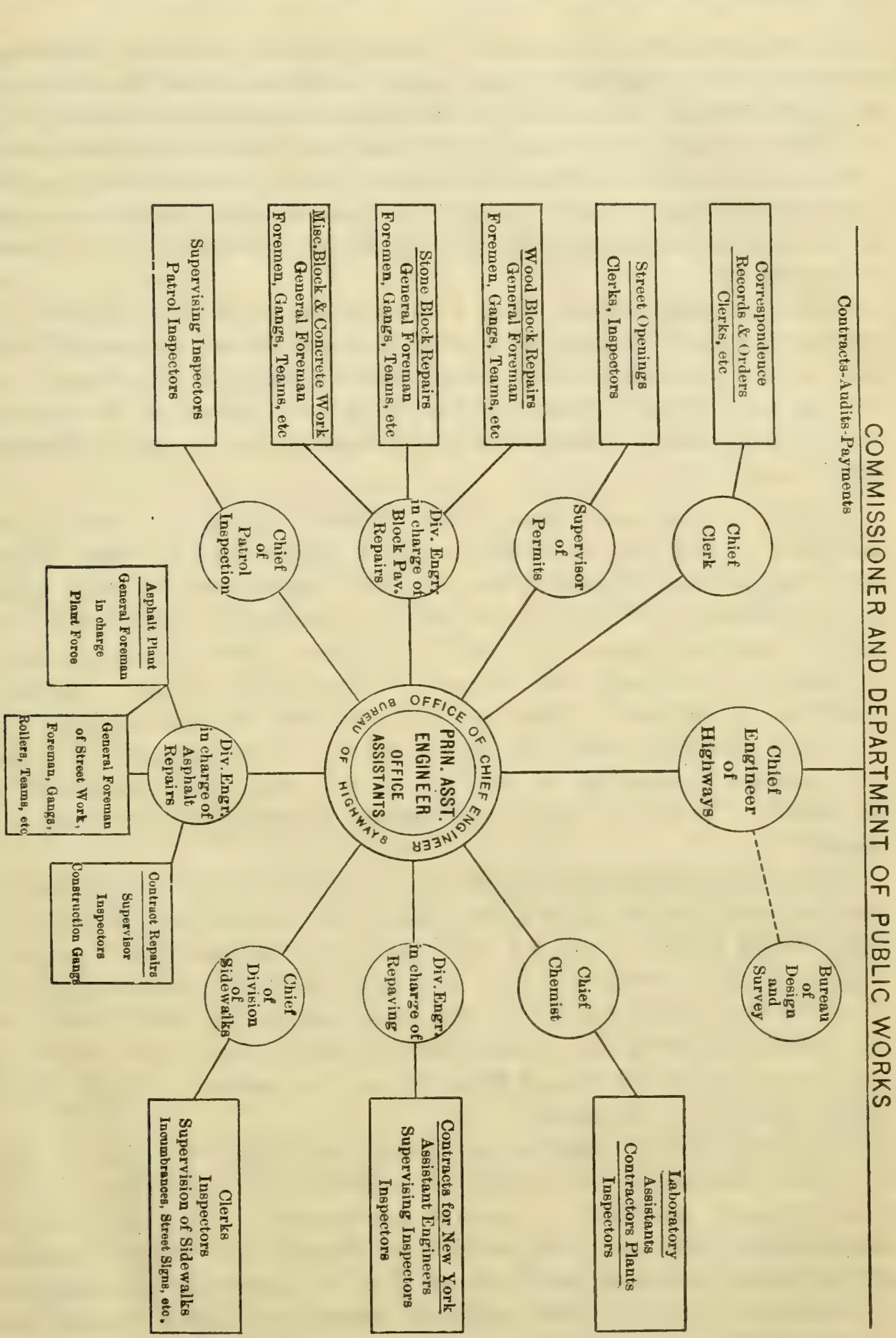

FIG. 2.

r. ...am. cation requirements is essential. Hence, one of the most necessary divisions is the laboratory and plant inspection force in charge of the chief chemist. Losses due to mistakes or inadequate inspection in this division may amount to many times in value what would occur from omissions or carelessness of inspection on construction, and the city will save many times the cost of a proper organization to supervise the manufacture and delivery of proper materials of all classes for pavements.

With an organization such as has been outlined, and with the proviso of the enforcement of proper co-operation on the part of other departments and sub-surface corporations, and the co-ordination of their plans in general harmony for the greatest good of the greatest number, it is believed that street surface maintenance can be efficiently carried on.

Ir. ...ker. ROBERT A. MEEKER, Esq.*—Mr. Whited having so clearly and succinctly stated the necessity for and importance of State Highway Engineering Organization, further arguments on that point are unnecessary. The keynote of all successful organization was struck, however, when he said "all must work harmoniously"; no matter how well the corps is organized, without harmony it will be a failure. Better a faulty system of organization with a united purpose than the most perfect organization in which each member is striving for his own glory, even at the expense of all the others. Then the result, no matter how perfect the design, is anarchy.

Appreciating this fact, an endeavor has been made to build up an *esprit de corps* in the New Jersey Road Department, the rallying cry being "Good Roads". Around that standard all gather with a harmony of purpose and a unity of action that have enabled the accomplishment of much that would have been otherwise impossible, and to-day even the farmer in the most remote district stands ready and willing to pay his share toward "Good Roads", and the motorist of the United States delights to use them.

The New Jersey organization, being the outgrowth of more than 20 years' experience, can hardly be classed as an experiment; it is, more properly speaking, a gradual development. Starting with one commissioner, the work was carried on through the county engineers of the several counties; as the work grew, a supervisor was appointed to assist him, and later two assistant supervisors and a chemist were added. Then, owing to the increasing demand for a higher grade of road, caused by the advent of the automobile, a larger organization was demanded, and the re-organization under the present Commissioner was the result. It consists of the Commissioner, who is chief executive, the

* State Highway Engineer of New Jersey.

Highway Engineer, who must pass on the design of all engineering work, four Division Engineers, three of whom have each charge of a prescribed district and the fourth has charge of the bridges, a Chemist, who tests all road materials, twenty-one county engineers, who, though employees of the county, are also resident State engineers in their several counties, ten inspectors, and six foremen; the latter are to form the nucleus of a force that will be needed when the State highway system becomes an accomplished fact. Mr. Meeker.

The foremen and inspectors report in writing weekly to the Division Engineers, who in turn report to the Highway Engineer and he reports to the Commissioner, thus each has his alloted task and each his share of responsibility. The county engineers might properly be classed as brigade commanders having command of the work in their several counties and being subject only to general orders from headquarters as to standards that must be met.

The State highway system, just referred to, is also a natural outgrowth of the work, its aim being to unite the several county seats and to extend the main highways to the borders of the State. This being such a large proposition, it was deemed wise to establish a Highway Commission which should determine the general plan and location of the work. This Commission consists of the Governor, the President of the Senate, the Speaker of the House, the State Treasurer, and the Commissioner of Public Roads. This Commission has laid out a system of roads 1 500 miles in length, most of which are already improved, hence the State's principal task will be one of maintenance, which, owing to the fact that New Jersey lies between the two largest cities on the Atlantic seaboard, will always be a big one. Owing to the fact that many of our main highways have been laid out for from 100 to 200 years, their exact location is often hard to determine, hence it was necessary to add a right-of-way engineer to the force, and his services have proven so valuable that, in all probability, others will have to be added.

In closing, a word of tribute to the contractors should be added, for to them the engineers owe much; it is their practical skill that makes the designs and plans of the most perfect engineering organization possible.

Paul D. Sargent, M. Am. Soc. C. E.—The one argument that has been most effective in every State in bringing about the creation of a State highway department has been incompetency and inefficiency on the part of local officials in securing economical results in the expenditure of highway funds. Consequently, every State highway engineering organization, to prove its right to exist, must be developed along the lines of efficiency, competency, and honesty. Politics, in the ordinarily accepted sense of the term, can have no place in such an organization. Mr. Sargent.

Mr. rgent. It seems to the speaker that the best results will be obtained and there will be more general satisfaction when the work is in charge of a commission of three members, provided they are men of sufficient breadth of view to understand their problem in its largest sense.

The commission should formulate the general policy to be adopted by the department, unless this has already been covered by law, and should pass on all large questions of a business nature.

Unless the commission is in daily session, which is rarely the case, it will be necessary for it to select an executive officer, who will usually be the chief engineer.

Besides engineering ability, this man should possess a good degree of tact, for meeting and dealing with the public, especially local officials. This is particularly desirable where highway improvements are paid for jointly by localities and the State for this always means that the State directs the work, and, unless handled just right, local officials will feel that their authority is usurped. Other things being equal, tact in dealing with local situations should be a controlling factor in selecting subordinates all through the department.

The chief engineer should have absolute control of the appointment of all subordinates, and, of course, they should report to him. The size of the organization will depend entirely on local conditions, that is to say, the quantity of work to be done in a given time and the area over which the work is distributed.

The following organization seems to the speaker to be about as small as can be expected to handle efficiently any considerable quantity of work: A division of surveys and plans; a division of design and construction; and a division of maintenance.

The division of surveys and plans will make all surveys and secure all preliminary data necessary for a proper design of contemplated improvements. This division should furnish plans of the road as it now is to the division of design and construction, where, after examining the locus, with plans, profiles, and sections in hand, changes of alignment and grade and details of construction will be determined.

The plans will then go back to the division of surveys and plans for completion, including preliminary computations of quantities, on which an estimate of cost will be prepared and bids will be invited.

At the same time the division of design and construction will prepare specifications, advertisements for bids, contracts, etc.

After the award is made and work is in progress, the supervision and inspection will be done by the division of design and construction.

It can be readily seen that in a small organization one man might handle both of these divisions, as their work is intimately correlated.

The division of maintenance, as the name implies, will look after this work especially, and it is suggested here, entirely on the suppos-

tion that both the other divisions are fully occupied with their particular problems. Mr. Sargent.

This whole scheme is based on the supposition that there will be no division offices, as there are in several State organizations, but that all engineering work is done at department headquarters.

To complete the organization, there should be a division of tests and one of accounting. Tests should by all means be reported to the chief engineer through the division of design and construction.

When the chief engineer is the executive officer, the division of accounts should report to him, in order that he may be informed of the fiscal operations of the department.

When a State highway department is in charge of a single commissioner, and a competent civil engineer is selected for the position, the scheme of organization herein outlined will be applicable by substituting in this discussion, the word "commissioner," for "chief engineer."

W. W. Crosby, M. Am. Soc. C. E.—In an efficient organization for producing physical results from men and materials, the responsibility for the results must be clear—if possible, individual—and the authority and resources in any case must be sufficient, not only to equal or balance the responsibility, but also, in order to be on the safe side and to avoid any arguments in the matter, to be appreciably in excess of the required duties and responsibilities. Mr. Crosby.

In highway work, a division of the consideration is frequently made between Construction and Maintenance, and, perhaps, naturally, up to comparatively recent, if not actually the present, time, the bulk of the investigation and discussion in the United States has seemed to center under "Construction". It is beginning to be realized, however, that "Maintenance" is worthy of at least equal consideration, and that the proper organization of the maintenance forces is of prime importance.

The speaker, after long experience and wide observation of the experience of others, has had at least one conclusion clearly crystalized therefrom. That is, that, not only for the sake of securing prompt satisfaction in the results of the expenditures for labor and materials toward maintenance, but also for the sake of economy and efficiency in the long run, it is necessary to separate the maintenance work from the construction, and thus clarify the responsibility for the proper performance of each.

Naturally, there is no need for a State Commission or Commissioner for the Construction and another for the Maintenance, nor of two Chief Engineers on these lines; the segregation may properly begin immediately below the Chief Engineer, or it may not begin until the Division Engineer, or even the Resident Engineer, is reached. Local

Mr. Crosby. conditions and due regard for the rest of the whole organization of the department will decide this point. Ultimately, however, the separation of work and responsibilities must be reached.

The speaker has worked up through the matter from the beginning, of all construction work and no maintenance, to the point where the maintenance mileage far exceeded that under construction, and he has tried the various methods of placing the responsibility for the maintenance on the construction engineers, or maintenance engineers, and on others. He has had the maintenance work done by patrolmen, by gangs, by contract, etc.; but satisfaction has been approached only through such arrangements as made the maintenance work and responsibility for its proper performance the sole duty of the employees up to a certain point where dependence could be had on that individual's appreciation of its proper relation to the importance of construction, and on whom the construction demands were not so great as to incite any neglect of the maintenance.

As has been previously stated by the speaker:*

"The importance and economy of prompt, efficient and sufficient maintenance cannot be overestimated in road administration, and this is especially true in the case of modern roads under present traffic conditions. More of the existing defects in roads today, whether such defects be those of appearance, of comfort, or of economy, are due more to weak points in maintenance than to deficiencies in construction or to any other cause.

"The requirements of maintenance work demand the careful performance of little things and the prompt attendance to such; persistence and continuity of action; good judgment and craftsmanship in actual work; and a high regard for economy and the old fact that 'many a little make a mickle'. Maintenance work contains far less of the spectacular than does construction, and thus some other stimulus is frequently needed by the workers in it that interest may not flag. Without some such incentive, the results are almost sure to be unsatisfactory."

Another matter to which the speaker wishes to call attention, in connection with the discussion under Organization, is that of the duties of resident engineers and inspectors.

The differentiation between these two cogs of the machine does not seem to be always as clear as it should be, and frequently, from lack of such clearness, duties seem to be extravagantly imposed on one or to be loaded on the other so as to cause a failure in the operation.

A resident engineer is generally supposed to be a man of some experience and skill in the work under his charge, and who, on account of such expertness, is delegated from above certain (limited) authority for making decisions and for directing the details of his work so as to facilitate the progress of the latter and relieve his superiors from the

* Report to the State Roads Commission of Maryland, 1912, p. 101.

necessity of considering and deciding minor details when once general directions have been issued in one way or another by them. It should be noted that ability, experience, firmness, good judgment, and tact are primary requisites for a satisfactory resident engineer. Mr. Crosby

An inspector, on the other hand, is merely the eye of his superior, to whom he reports, focussed on the particular work on which he is stationed for the purpose of observing or inspecting its performance. He is instructed, by means of copies of the clauses of the specifications that apply to that work under his observation, or by other means, as to what is required for satisfaction of the contract in such work, and, in case of any apparent evasion or violation of the provisions of his instructions, is or should be directed to call the attention, first, of the contractor and, secondly, of his nearest superior with authority to the matter.

An inspector, as such, has or should have no authority over the work delegated to him, and his responsibility is, or should be, accordingly limited to reporting promptly on the facts as he notes them. To delegate authority to him makes him no longer an inspector but rather a resident engineer of more or less limited field, as the case may be.

It is not the speaker's intention to provoke discussion of the etymology of the two expressions, or to quarrel with their various uses, but rather, by the foregoing, to illustrate differences of meaning that may be assigned to each for the main purpose of clarifying and, as far as possible, lending force to the following remarks.

The speaker (and doubtless those to whom these remarks are addressed) has continually heard references to difficulties supposed to be omnipresent between contractors and engineers, and much time on many occasions has been occupied in discussing such differences or sources of friction and means for their reduction or relief.

From a long experience, in capacities ranging upward from inspector to chief engineer, the speaker has become convinced that the responsibility for such friction rests largely in many cases with the chief engineer or other party responsible for the organization of the engineering forces for highway work.

In connection with the actual organization of the engineering forces, consideration of the specifications for the work itself must not be neglected, for, in part at least, they generally affect the operation of the machine or express in words what is graphically or mechanically shown by the form of the organization itself.

In a lecture at Columbia University a year ago, the speaker made the following statements which it seems to him are pertinent for repetition here:

"With a proper organization of an engineering department, including the definitely understood delegation of authority, the full expres-

Mr. Crosby. sion of the work to be required by plans and specifications, the location of the responsibility on the proper party (the chief engineer) and this recognition of their obligations on the part of the contractors, ninety-nine per cent. of the present causes of friction between the latter and the engineers would be removed and in place thereof would be substituted the cordial assistance of the engineers to the contractors.

"To hitch up an experienced contractor's foreman, bent only on making money for his employer, on one side of the pole with a young, ambitious but green, inspector (perhaps the only one obtainable) on the other side, and then to turn this team loose and to expect them to keep steadily along the road toward success would be folly, yet it is often attempted, for valuable inspectors do not often remain as such as long as do foremen; but if the foreman realizes that the inspector is to call for help the moment he feels he is losing his footing and that the call will be answered by an experienced man fully capable of handling the situation, there is far less likely to be the need for such a summons. Few competent engineers and contractors cannot get along together peaceably and advantageously. The difficulties seem to come largely from mismatching, by improper delegation of authority, and from lack of clearness and definiteness in describing the work and materials to be furnished.

"The speaker knows from experience the difficulties of the young, enthusiastic, impatient resident in preserving a placid equanimity day after day when constant evidence of a desire on the part of a contractor to evade the specifications, to substitute 'something just as good' for the materials and work he has agreed to deliver, is offered. On the other hand, he sympathizes with the contractor who is constantly being nagged by an inspector over minor deficiencies, or when the inspector tries to 'run the job himself'. Perhaps it is human nature for most contractors to try out an inspector to see what shortcomings he will quietly submit to, and, having found out, to his advantage or loss as may be the case, thereafter to gauge his work accordingly. Perhaps it is natural for most contractors, after competitive bidding on public work, to attempt to substitute at every possible opportunity 'something just as good' for the article specified. But the speaker believes that much improvement in the relations between contractors and engineers would be had were the organization and specifications, as before referred to, made most explicit and then the contractors to regard their carrying out to the letter as much an obligation on themselves as the completion of the work."

Therefore, in framing an engineering organization for highway work, and in drawing the specifications for the work under it, the speaker believes that full consideration should be had, previous to arranging for the resident engineers and the inspectors, of the possibilities of securing competent "residents" for the salaries fixed to be paid; of the possibilities of securing alert and satisfactory "inspectors" for the salaries fixed for these positions; of the competency and character of the contractors likely to carry on the actual work,

and of other conditions likely to affect the decision, and then to design the machine in all its details so as best to fit all the conditions. Mr. Crosby.

It is scandalous for an engineer to design a machine requiring steel in all its parts and then, on that design, to build it of wood because wood happens to be the available material. It will be disastrous to allow the use of wood for a cog when steel is known to be needed, and every cog in the machine, even the least important, should be recognized as needing in its character an ample "factor of safety".

One word further: Such a machine should not be designed or built too delicately or too lightly. The larger and more complicated the machine, the more necessity there is for sufficient mass in the machine parts themselves to take up the shocks and vibrations, from both without and within, incident to its operation. Mass is necessary for momentum, and a sufficient momentum is desirable in this case for the purpose of continuing action in spite of the inevitable obstructions in regular and efficient operation.

FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

P. E. Green, M. Am. Soc. C. E. (by letter).—One of the rules adopted for this discussion is that the matter to be presented must be new. It is a very proper rule, but it should be realized that it is a very difficult one to live up to. As far as new matter is concerned, it is hard to conceive of a phase of this subject which has not been touched upon or investigated more or less thoroughly, except possibly from the laboratory end. It would appear necessary, therefore, to confine oneself to an academic discussion of the matter or to present individual experiences showing some variation from expected results. In order to bring out a broad line of discussion, it might be well, however, to systematize the factors entering into the problem and call attention to some phases which might be investigated a little more than has been done in the past, with the hope that some important new points may be presented. In opening this subject the writer will confine himself to a general outline of the problem, leaving the details to be discussed by those who follow. Mr. Green.

The subject as assigned may be logically divided into two interlocking general sections, namely: "The Selection of Materials", and "Methods of Construction". Each of these main headings may be further divided into numerous and important parts. All these parts should be taken into consideration by any public authority, such as a city, county, or State, when it is proposed to enter on a general scheme of highway improvement such as is so common at this time. In this way it is not difficult to make a logical selection of the various materials and methods involved in the construction of a system of streets or highways; and a scheme or plan of future construction

Mr. een.

may be laid out. This does not mean that only one material shall necessarily be selected for any one street or highway. It simply means that the class or type of material shall be selected, and that choice must ultimately be made between several more or less suitable products.

A diagrammatic outline of such a general scheme covering the subject of this discussion is herewith submitted. It is not claimed that it is particularly novel or complete, but is presented for discussion and improvement. Although intended primarily for country highways, it may be used with equal facility for city streets.

DIAGRAMMATIC OUTLINE OF "FACTORS" IN ROAD WORK.

- Highways under Consideration.
  - I.—Selection of Materials.
    - 1.—Character of Road (City or Country).
      - Trunk.
        - Through traffic road—commercial or passenger, or both.
        - Commercial road, but not necessarily through traffic.
      - Secondary. Local.
        - Connecting important centers to trunk lines.
    - 2.—Traffic.
      - Much light fast passenger; or
      - Heavy hauling; or
      - Only local hauling.
      - Roadway widths required.
    - 3.—Climate.
      - Northern.
      - Intermediate.
      - Southern.
    - 4.—Character of Supporting Country.
      - Property values and benefit to be derived.
      - Vicinity of large cities.
      - Topography—drainage.
      - Character of business using road—esthetic considerations.
    - 5.—Materials.
      - Class.
        - Superior.
        - Inferior.
        - Physical conditions governing, such as wear, crown, dust, sanitation, etc., etc.
      - Cost—maintenance charges.
      - Character and appearance.
      - Ease of handling materials.
      - Relation of material to roadway width.
  - II.—Methods of Construction.
    - 1.—Laws and legal organizations governing.
    - 2.—Location of railways with respect to improvements—railway rates.
    - 3.—Existing good roads as affecting haul—wagon or truck.
    - 4.—Availability of labor.

The scientific selection of the material to be used in the construction of a highway depends on all the items named in the diagram. The diagram itself looks somewhat complicated, but its use is very simple. Before commencing any large scheme of highway improvement, the outline may be studied with reference to materials, methods, etc., and simple tables prepared. Too often some essentials are not considered seriously or are forgotten, and for this reason much money, time, and labor are wasted. From time to time, somewhat similar schemes have been advanced, but frequently they deal with only one essential, such as traffic, to the exclusion of other important elements. For example, in addition to traffic, the character of a city highway should depend on the layout of the city, its topography, population, business, education, and wealth; and, further, the location of the city as regards the availability of standard materials, such as brick, wood block, crushed stone, gravel, sand, tar, asphalt, etc., etc., is vitally important. Mr. Green.

Considering the problem of a country highway system, many entirely different factors enter into the selection of the material, such as vicinity of cities, large or small; general population and wealth of the surrounding country; whether or not the highway is to be a trunk line for through traffic, and if so, whether such traffic is light, high-speed traffic, or heavy, low-speed, commercial traffic.

If the highway is a secondary one, as regards the through traffic, it still may be a trunk line as regards its commercial traffic, such as a road leading from a trucking district to any one of our large cities. Such a highway may be comparatively a short one, yet the character of its traffic actually makes it a trunk line.

The least important class of country roads is generally referred to as "local roads". These are exactly what the name signifies, and whether or not they are to be improved depends on the wealth and education of the surrounding country and its inhabitants. Such roads are generally paid for only by local taxation.

The effect of "climate" is more important than is often considered. Some pavements, such as the various bituminous pavements, concrete, etc., are very much affected by the temperature. Thus a surface mixture of asphalt suitable for Chicago is not suitable for New Orleans; and concrete as a wearing surface should do better in Houston than Buffalo.

Three zones may be recognized: Northern, embracing the New England States, New York, Pennsylvania, Ohio, Illinois, etc.; the Intermediate, such as Virginia, Kentucky, Tennessee, Arkansas, etc.; and Southern, such as Alabama, Mississippi, and Texas, which have little winter. In the Northern States, the selection must be of materials not affected by extremes of temperature, freezing and thawing. For the Intermediate, it is not necessary for the materials to withstand great variations in temperature. For the Southern, materials which

Mr. Green. might utterly fail in the North because of inability to withstand extreme cold or great variations in temperature, may be very satisfactory. Furthermore, the exceedingly heavy and continuous rains following long hot, dry spells in the southern sections must be taken into account.

A great deal of money might often be saved by considering carefully the various classes of paving material, which may be suitable for the same purposes and yet may vary widely in cost in the different parts of the country. Thus, wood block pavement may be a very reasonable one for the business streets of small cities in regions close to the sources of supply, but for other cities, similar in size, but not so situated, it would be a grave mistake to use it, because of its prohibitive cost. This same wood block pavement has a characteristic in common with sheet asphalt: it must receive a certain minimum traffic or it may fail. On a wide roadway, having only a traffic along the center, the blocks outside this used strip will curl up under the sun and swell and "blow up" if a rainy fall follows a hot, dry summer.

Further, such a material as trap rock, though making good macadam highways in New York, probably better, as far as wear is concerned, than any of the limestone used in the West or South, would be impossible for these last locations. Yet the writer has seen specifications, for roads hundreds of miles away from granite quarries, which called for granite top macadam, though local limestone at half the cost would wear nearly as well. Again, there are sections of the country where the high cost of cement, but the comparatively low cost of good stone or gravel, makes a concrete base for a pavement quite expensive, but a crushed stone or a natural gravel rolled base would cost less and be nearly, if not entirely, as durable. Of late years, too little attention has been paid to the macadam base as a foundation for a durable pavement. The public has been educated to think that concrete is the only possible foundation. The situation is similar to that of the septic tank in sewage disposal. Many public officials have heard and believe that such a tank turns sewage into drinking water.

In those parts of the country in which there has been much road work and improvement, it is much easier to get competent foremen and good labor than in sections where little, if any, of such work has been done. The result of good workmanship is very apparent. The local officials, also, are better educated in these matters and more inclined to give an engineer sway.

It should be remembered, also, that the character of the highways, whether city or country, must depend largely on the ability of the tax-payers to pay for them. A sparsely settled region may be very desirous of having good roads and be willing to be taxed for their construction, within reason, but Wisconsin, for instance, is far less

able to pay $10 000 per mile for highways than is Massachusetts. The road mileage depends only to a limited degree on the population. Mr. Green

There can be little question of the benefit derived from the construction of good highways, but the degree of immediate benefit should have much to do with the selection of the material for such improvement. It might be, and frequently appears, that the benefits in the long run from a thoroughly high-class permanent construction will, theoretically, far exceed those derived from any cheaper construction. Because of the large immediate benefit received from the cheaper construction, however, and because the tax-payers are unable to pay for the more expensive construction, it may be much better finance to put in the cheaper highway, and thus not over-burden the community. Future development and growth of wealth may provide equitably for the construction of a better improvement. Is it not the same principle that governed the construction of our early railroads? It is a real recognition of this principle, not altogether possessed by some engineers, that has justified in many cases the voting of long-term bonds for short-lived improvements. Such a principle must be used with caution, however. It cannot be defended in New York or Illinois, but certainly can in Texas and Oklahoma.

In addition to the various points noted thus far, the esthetic features of the wearing surface of a highway should not be forgotten. In many cases, this feature should be made a very large factor in the selection of the proper material. Thus, a smooth, well-built brick pavement might be nearly as noiseless as one of bituminous concrete, and might have a much less maintenance charge and be more sanitary; but, along a scenic highway where the adjoining country is very highly developed esthetically, such as in the vicinity of a large city where there are many great estates, it is quite probable that the bituminous concrete pavement would be considered more suitable, pleasing, and satisfactory by those who pay for it. Such a road would be used largely by automobilists and comparatively little by truck gardeners. If this highway were lined by truck farms, however, could there be any doubt that the selection, if made between the two, should be brick, especially as it has been demonstrated that the bituminous concrete does not withstand iron-tired traffic? This selection, moreover, would be the correct one, even if the pleasure traffic was very large.

One of the points which it is believed has been neglected in the past is the relation of traffic to roadway width, especially where traffic is dense. It seems to the writer that it is possible to establish a logical, if empirical, formula which would be a considerable help at times in the solution of this vexatious problem. Some years ago, in preparing a report on a popular scenic highway near Chicago, a traffic census was taken at various points, and in addition statistics on other popular

park highways in Chicago were obtained. These were plotted, together with the various existing roadway widths, and several curves tried on the same sheet. In this manner, it was finally decided that a parabolic curve having the equation:

$$Y = \sqrt{\frac{X}{6}} + 16,$$

would express the desired relation. In this equation, Y = the roadway width, in feet, and X = the number of vehicles.

Thus, if it is determined from a traffic census that within 5 or 10 years a highway will have a probable traffic of 2 000 vehicles, the formula would give a roadway width of 34 ft., as shown on Fig. 1.

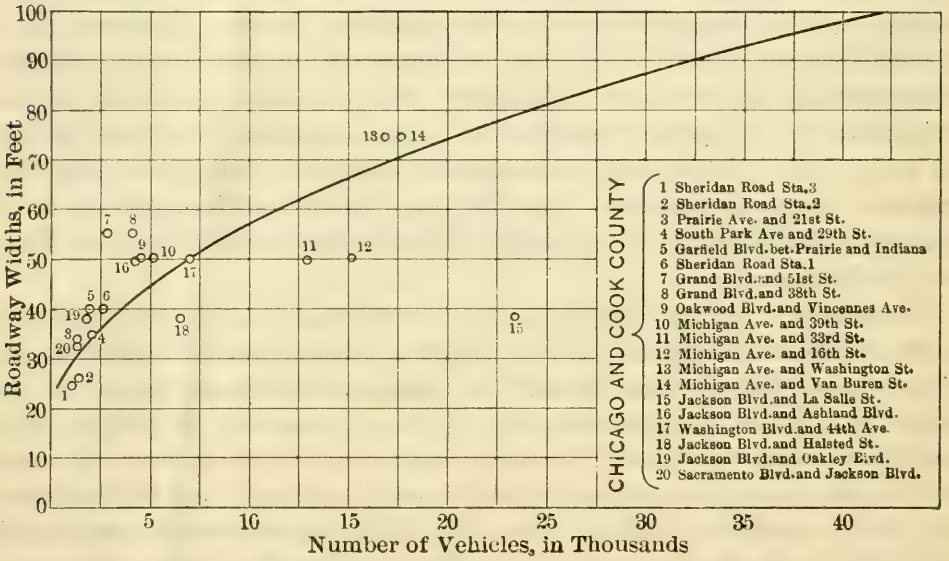

FIG. 1.

The diagram, or formula, is of course faulty as it takes into account only two factors, but it has been a help in arriving at conclusions at times, and is offered for what it is worth.

Attention would be called to the fact that, because it is decided that the roadway is to be of a certain width, say 34 ft., it does not follow that the entire road should be paved with an expensive material. Cuyahoga County, Ohio (Cleveland), has hundreds of miles of roads improved on one side with brick and on the other with plain macadam or gravel. The empty wagon and light horse traffic prefer the latter. Winona County, Minnesota, has paved the central 8 ft. of some of its highways with concrete, and provided wide macadam shoulders.

E. H. THOMES, M. AM. SOC. C. E.—An important factor in the selection of materials would be a standard specification for gravel, broken stone, etc., so that there would be no misunderstanding as to the sizes referred to; it would also be to the advantage of all buyers

and sellers of broken stone. At present, this material is specified by minimum, maximum, and average lengths or diameters, by passing through a ring in any one or all directions, by the diameters of the screen perforations, by arbitrary designations, and in various other ways. Some designate No. 1 stone as the first course laid, which is usually the large stone, and others designate No. 1 stone as the small stone which first passes through the stone screens. Some refer to nut size, which may be any size between those of a hazel-nut and a cocoanut. The speaker presented this matter to the Association for Standardizing Paving Specifications 3 years ago, and also in more detail to one of the road meetings of this Society 2 years ago, but very little has been accomplished since. His attention was called to this subject at the International Road Congress in London in the summer of 1913 by a report of the work done by the (British) Engineering Standards Committee on the standardization of road material. The report was published in the daily official journal of the Road Congress at London, but very little mention of that work has been made in the United States. Mr. Thom

The Engineering Standards Committee is supported by the Institution of Civil Engineers, The Institution of Mechanical Engineers, The Institution of Naval Architects, The Institution of Electrical Engineers, and the Iron and Steel Institute, all of Great Britain. It has issued more than sixty reports and standard specifications, mostly along mechanical and industrial lines. In the summer of 1912 a Sectional Committee on road material was formed. It recognized the need and importance of the standardization of broken stone, and that was the first matter taken up. A sub-committee of the various interests was appointed, and, after considerable discussion and investigation, the British Standard Specifications for sizes of broken stone and chippings was adopted and issued by the Engineering Standards Committee in the summer of 1913.

Stones 1 in. in diameter and less are designated as chippings. There are four standard gauges of broken stone, $1\frac{1}{2}$, 2, $2\frac{1}{2}$, and 3-in. Broken stone specified as $1\frac{1}{2}$-in. gauge shall all pass through a $1\frac{1}{2}$-in. ring, and shall consist of the following percentages by weight: Not more than 15% passing through a 1-in. ring in every direction; not less than 65% more than 1 in., and not exceeding 2 in. in greatest length by measurement; not more than 20% greater than 2 in. in greatest length by measurement. The specifications for each $\frac{1}{2}$-in. larger gauge are similar to the foregoing, with $\frac{1}{2}$ in. added to each dimension stated, excepting the 3-in. gauge where 4 in. greatest length is used. Six standard gauges of chippings are recommended, namely: 1, $\frac{3}{4}$, $\frac{1}{2}$, $\frac{3}{8}$, $\frac{1}{4}$, and $\frac{1}{8}$-in. All 1-in. chippings must be capable of passing through a hole 1 in. square; and at least 70% by weight must be retained by a sieve having holes $\frac{3}{4}$ in. square. The specifications

for the other sizes are similar, with the substitution of the sizes of sieves. With $\frac{1}{8}$-in. chippings, the sizes of square holes on which the material must be retained is $\frac{1}{16}$ in.; everything passing that sieve is to be called sand. They also specify gauge plates, sieves, and methods for testing fair samples of at least 100 lb. of the material. It will be seen that in Great Britain the broken stone is divided into more sizes than is customary or advisable in the United States. Some of the quarries there have been developed much longer; they are down deeper, and in sounder and more uniform rock. Some of them also furnish washed chippings, which are advantageous for bituminous work.

The materials and quarry practices in the United States are similar to those of Great Britain, and, with the work already done by the Engineering Standards Committee, it will be an easy matter to obtain a standard specification for sizes of broken stone. The question must be considered from the standpoint of both sellers and users, for all purposes. Conditions here seem to warrant that this subject be taken up by this Society, and in order to secure some definite action on this matter, the speaker will move that the Board of Direction be requested to appoint a Special Committee on the Standardization of Broken Stone, to report to the Society as soon as practical.*

S. Whinery, M. Am. Soc. C. E.—The diagrammatic outline of factors in road work submitted by Mr. Green is generally a very satisfactory classification of the elements entering into the design of highways. The speaker, however, would remove the element of esthetic considerations from the subdivision under "Character of Supporting Country" and give it a separate sub-head at the end of the second column. It may be questionable, however, whether this item should have any place in such a diagram, devoted as it is to what may be called the economics of highways. Esthetic considerations, of course, cannot be ignored finally, but, from the engineering point of view, they are foreign to the main problem, which, in highway design, as in other engineering structures, may be reduced to the question: What design will best serve the purpose intended, with due regard to the highest yield in returns on the money invested? Necessity, of course, may force us to substitute the question: How may the highest economical results be obtained for the money available? Esthetics is not a factor in solving either problem.

Nevertheless, it must be finally considered, and the answers to the foregoing questions may have to be modified accordingly. The highway should be designed along the same lines as any other industrial

* This matter is referred to the Special Committee on Bituminous Materials for Road Construction, with the request that some action be taken to secure the standardization of broken stone in the United States.

structure, as, for instance, a building for manufacturing purposes. Here the efforts of owner and architect will be first directed wholly to questions of utility and economy. These settled, the question of the adaptation of the design to artistic ideas or standards will be considered, and such modifications or additions as may please the designer, or as the owner may feel willing or able to afford, will be incorporated. Mr. Whine

In any event, the question of esthetics, however important a factor it may be considered in designing city streets, boulevards, and parkways, is usually so unimportant in as far as it relates to the roadway itself, in the case of country highways, as to be almost negligible.

The speaker cannot agree with Mr. Green that the building of temporary, inadequate highways is justifiable, except in rare instances, especially where long-period bonds are issued to pay for them. In the present well-developed status of scientific highway construction and economics, and the possibility of securing capital for enterprises of real merit, the spending of the public money for highways of temporary life and usefulness is generally inexcusable. Mr. Green's citation of early railroad history does not present a parallel case. The temporary and inadequate railroads built in the early days resulted from the undeveloped conditions of the principles and requirements involved, rather than from financial or economic considerations. Intelligent and reasonable people do not pattern after them now, to the extent of adopting an ephemeral and clearly inadequate construction, which must obviously be abandoned and reconstructed in a few years. It would be more difficult to finance such an enterprise than one properly designed to serve adequately the prospective requirements of the reasonably near future.

As a business proposition, the speaker does not believe that it is wise or justifiable to issue long-period bonds to pay for a structure the useful life of which will be but a small fraction of the period the bonds are to run. In other words, we have no ethical right to saddle upon posterity a burden from which posterity will not be benefited. From the sound business standpoint, a highway improvement the benefits from which will not redeem its cost during its life, is not a justifiable investment for a community or a State.

This, however, is not an argument in favor of unnecessary or reckless expenditure in the building of a highway. No more money should be expended on it than the conditions or needs of a community, or the general public, now economically justifies or is likely to justify during the life of the improvement. The strict application of this criterion would rule out the costly improvement of many ordinary highways where a properly constructed dirt road is all that the conditions warrant.

The considerations relating to selecting the type of roadway and the materials to be used in its construction, submitted by Mr. Green, are

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in the main apropos and sound. It would have been appropriate for him to add a warning against the use of materials, merely because they are of local occurrence, or because of their cheapness, and a strong protest against the use of materials or methods the utility of which has not been sufficiently established by experience. The use of experimental materials and methods, except on a very small scale for trial, ought to be discouraged by the prudent engineer. Progress demands, of course, that promising innovations from established practice shall be tried out, but the conservative engineer will prefer to let other people do the experimentation, except on a scale which will not result in any considerable loss to his clients, in case of failure.

Personally, the speaker feels that, in road and pavement work, we ought to adhere pretty close to materials and methods the value of which has been clearly established.

The more important country highways are now subjected to very much the same conditions of use as the average city street. Long experience, both in America and abroad, has, after thorough trial and much foolish and often disastrous experimentation, determined that certain types of pavement and certain materials, can alone be depended on to produce satisfactory results, and their use on city streets has become standard. Among these the choice, of course, will be influenced by local conditions. Prudence and good engineering would seem to dictate that we should apply this experience and established practice to the construction of modern roads. The common argument that the first cost of a standard roadway pavement is too great to warrant its use is in most cases fallacious or untenable. An established highway is, nearly always, a permanent structure, and should be designed with reference to its true economy, that is, its value as an investment. If, for instance, a standard brick-paved roadway will last 20 years where a water-bound macadam surface will last but 5, and the former can be maintained at a less annual cost, it will be, at ordinary prices, much the more economical in the end, and the engineer should make this plain to his clients. He should vigorously discourage the common disposition of the public to waste its money on temporary highway improvements because of their low first cost, and should encourage the building of roads in such a manner as will prove most truly economical. These are such trite arguments that they seem to need to be repeated until their truth is accepted and more generally acted on.

The speaker agrees with Mr. Green that the width of the paved roadway should be determined with reference to the density of travel to which it is likely to be subjected, but thinks that the tentative formula submitted for determining the proper width, when applied to country roads, should be used with even more caution than the word tentative implies. The data on which it is based seem to have been

derived from conditions on very heavy traveled streets, boulevards, and suburban roads, where the conditions are very different from those on most country highways; but, even in the examples used, there is such a wide variation in the figures as to suggest their uselessness as a basis for determining a standard of width of roadway. Thus, as nearly as can be determined from the diagram (Fig. 1), the number of vehicles per foot of width of roadway varies from about 35 to more than 600. Mr. Whine

It is doubtless true that the higher figure indicates dangerous crowding, but, even so, a roadway having only 6% as many vehicles per foot of width cannot be regarded as an appropriate factor in determining a proper and safe standard for allowable density of travel.

If the proper width is to be determined by this method, only observed data, from such highways as seem to have just about the quantity of travel they can safely and conveniently accommodate, should be used. Deductions or formulas from such observations would be far more useful and trustworthy than the theoretical considerations and conclusions that some have tried to work out and apply. It is very desirable that data from highways of the character mentioned should be collected and collated.

W. W. Crosby, M. Am. Soc. C. E.—Mr. Green is to be complimented on his interesting presentation of this subject and on his exposition of the factors involved in the selection of the materials or the methods of construction. His diagrammatic outline of factors in road work is most interesting, and if one will take the trouble to compare this diagram with that submitted by the speaker 3 years ago,* an interesting illustration will be afforded of the ideas in common as well as those of difference which may be held by two individuals. Mr. Crosb

The speaker has been particularly interested in the scientific solution, proposed by Mr. Green, of that troublesome problem, the determination of the proper width of a roadway, and desires to express his belief that it is this sort of treatment of many of the present questions in highway engineering that will do most to advance the status of this particular branch of the profession and to erect for its future growth a substantial foundation.

In reference to this matter, and with the hope of furnishing data for the basis of such treatment of some of the problems connected with the selection of materials and methods in highway work where the traffic factor seems preponderating, the speaker offers the following concerning traffic records in general and in a particular instance coming under his observation:

In 1903 he urged highway engineers to take traffic censuses and study the relations between traffic conditions and the life, or ex-

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 6.

pense for maintenance, of road crusts. He also suggested that from the study of such relations much might be learned that could be used advantageously in making selections for construction under known or determinable traffic and other conditions.

The Illinois Highway Commission began this traffic census in 1906. In 1909, the Massachusetts Highway Commission took a census of traffic on Massachusetts roads.* Other traffic censuses have been taken and reported by officials and individuals from time to time since these dates, and though at first there was some skepticism as to the value of such investigations, the speaker believes that there is now a fair amount of general agreement that, with proper deductions made therefrom, they are of great value. He also believes that there is general agreement now that a road crust satisfactory and economical for sustaining rubber-tired motor traffic may not be so, either for horse-drawn or for hard-tired traffic, and *vice versa*, as he has stated previously.

During the summer of 1913, in order to permit the improvement of a highway by the State Roads Commission of Maryland, the Board of Park Commissioners of Baltimore opened to all kinds of traffic a park road paralleling the Reisterstown Turnpike for a distance of 2 500 ft. Previously, this park road had been kept in excellent condition by the maintenance, on the quartz-schist macadam, of a bituminous (or "pitch") surface, at an annual cost of about 9 cents per sq. yd. Within 90 days after the opening of the road to all kinds of traffic, this surface or "carpet" became in dry weather a dusty mass of loose material and in wet weather a mess of slimy black mud an inch or more thick. Depressions in the macadam beneath also began to show soon afterward, and by December, 1913, the road crust appeared to be generally going to pieces. The quartz-schist had shown a rattler test of about 4, though its cementation power, in use, had proved to be high—probably due to its iron content.

Southward from this 2 500-ft. stretch, the pleasure traffic continued along the main road through the park, the commercial traffic, which consisted mainly of horse-drawn, hard-tired vehicles and motor trucks, being turned out of the park at this point. The traffic on the 2 500-ft. stretch before its opening to commercial vehicles was therefore equal to that now using the road southward from this stretch, as shown by the census under "B" in Table 1; the combined traffic on the 2 500-ft. stretch is that shown under "A".

The carpet referred to in this case was made up by successive applications, in 1911, 1912, and 1913, of "75% Asphalt Oil", the layer of each being covered with pea gravel or stone chips, and at the time of its first subjection to the new traffic it was about ¼ in. thick on top of the macadam. The last application of bituminous material (or "pitch compound") and stone chips was made the day before opening the

* Report, Massachusetts Highway Commission, 1909, p. 128, etc.

road to mixed traffic. The width of the carriageway, exclusive of gutters, is 22 ft. Mr. Crosb

The figures of the traffic census taken in 1904 are inserted in Table 1 in order to give an idea of the increase or change in character of the traffic between that time and the present. They also give some information as to the amount of traffic under which a water-bound macadam of comparatively soft broken stone may be maintained satisfactorily at a not excessive cost. Such a road crust in this instance required treatment with a pitch compound before the traffic reached the figure in "A", in order to permit its maintenance with satisfaction at any cost.

TABLE 1.—Traffic Census on Main Road in Druid Hill Park, Baltimore.

Classification	Maryland factor.	November 24th to 30th, 1904.		November 20th, 1913.			
				"A"		"B"	
		Average per hour.	Units.	Average per hour.	Units.	Average per hour.	Units.
Saddled horses..........	1	3	3	10	10	0	0
One-horse vehicles.....	2	28	56	25	50	7	14
Two " "	4			35	140	0	0
Three " "	6	10	150	1	6	0	0
Four " "	8			0	0	0	0
Six " "	12			1	12	0	0
Total horse traffic..		41	209	72	218	7	14
Bicycles	2	2	4	8	16	6	12
Motor runabouts........	10	0	0	9	90	7	70
4-seat motor cars.......	20	1	20	43	860	39	780
6 " " "	40	0	0	37	1 480	35	1 400
Motor trucks...........	20	0	0	9	180	0	0
Totals...............		44	233	178	2 844	94	2 276

In making deductions from the data in Table 1, it may be of interest and value to refer to the similar data and the conclusions thereon,* furnished by the speaker a year ago concerning the Park Heights Avenue experimental work.

One further point may be indicated. It will be noted from the traffic tables that the increase in traffic on this section, due to the admission of commercial vehicles, was 204 units of horse-drawn and 364 units of motor traffic, the latter being composed mainly of motor trucks. Both these types have a high abrasive or crumbling effect.

* *Proceedings*, Am. Soc. C. E., for September, 1913, p. 1702 *et seq.*; or *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 138 *et seq.*

Evidently, the thin carpet over the soft macadam stones was unequal to the task of maintaining its integrity under this traffic, or of absorbing the shocks coming on it sufficiently to prevent the disintegration of the stones under it. Hence, under this traffic, the stones were abraded or broken to pieces, the bond of the carpet to the stable foundation was thus destroyed, and the carpet itself aided in going to pieces. The actual destruction proceeded along these lines, the carpet first scaling off in spots and revealing in these places a ground-up condition of the macadam surface beneath. Then the scales of the carpet, thus loosened, disintegrated, and finally the whole carpet went into dust or mud.

The experience here, as well as in some other cases with similar conditions coming under the speaker's observation, is quite similar to many instances of the failure of thin carpets on concrete roadways, and supports the theory that, with friable bases, the carpet must have such thickness and character as will prevent the disintegration, under the shocks of traffic, of the surface of the base itself, if successful maintenance is to be had.

Mark Brooke, Esq.* (by letter).—There is no one best paving material for all conditions, and the necessity of carefully considering in each case the many factors shown with such definiteness and detail in Mr. Green's diagram cannot be emphasized too much. As Mr. Green has stated, the use of that diagram in practice is comparatively simple, for it will be found that the choice in many cases will be restricted either by limitations of first cost or by local or general conditions which automatically cut out certain materials from consideration.

In contradistinction to the bridge engineer, for example, the highway engineer often finds that the sum of money placed at his disposal is not based on his own estimate of a definite project, but that he is allotted, in a general distribution of funds based on anything except a careful estimate or budget, a lump sum of money which he must spread over a certain section to the best advantage of the entire area. The question is then, not what is the best material, but how much he can afford to spend on a given street or road, so that the problem is apt to resolve itself into a selection of materials for the most suitable pavement which can be constructed for a definite limited sum, say, for $1.25 per sq. yd. Even in this narrow field of choice careful consideration should be given to all the items in the diagram, as far as the limitation of cost will permit.

Again, it will be found possible to simplify the selection of materials by applying once for all such of the factors in the diagram as have a general and permanent application in any given locality, thus elimi-

* Captain, Corps of Engineers, U. S. A.; Assistant to Engineer Commissioner, District of Columbia.

nating a large class of materials, and restricting the choice to a few types. Mr. Brook

In Washington, for example, as a result of experience and of consideration of the various factors such as first cost, character and volume of traffic, suitability of local material, esthetic considerations, etc., the problem has been reduced to a selection of one of three types of asphalt pavement for city streets, and bituminous concrete, cement concrete, or water-bound macadam with a surface treatment of light tar or oil, for suburban streets and county roads.

In the city practically nothing is used but a standard sheet-asphalt, a very limited quantity of 2-in. asphalt block, and an increasing quantity of bituminous concrete in accordance with the specification of the department.

This bituminous concrete, which is very similar to the various pavements of that type laid between 1870 and 1880, appears to have all the advantages and but few of the defects of sheet-asphalt, and is cheaper.

A gravel cement concrete base is now being used under all pavements, except in a few instances where old macadam roads have been surfaced with bituminous concrete.

Macadam, especially an old macadam road, will make an excellent base which will outlast any bituminous wearing surface yet devised, but it cannot be kept free from local settlements, nor can it be easily or economically resurfaced. The man who lays bituminous concrete on a macadam base should realize that he is storing up trouble for himself or his successor 15 years hence.

Whether or not it is advisable to invest in the more expensive base to reduce the cost of future resurfacing is, of course, a question every one must decide for himself.

The engineers in Washington for some time past have been of the opinion that it is better, as a general practice, to lay a base which will not have to be destroyed every time a new surface is put on it, and hope to relieve their successors of the extra expense and difficulties encountered during the last 10 years in resurfacing pavements laid on macadam and bituminous macadam bases—types of construction which conformed to the best engineering practice of the time, when the use of Portland cement was still in its infancy.

For 30 years a concrete base has been laid without either longitudinal or transverse joints, with no significant bad results, and the writer doubts very much the necessity of the elaborate and costly provisions for such construction which are so often seen in cement road work. On some of the cement pavements in Washington, no joints whatever have been provided, on others there is a thin transverse joint every 50 ft., made by bringing the concrete up to a vertical face, laying a strip of folded tar paper against this face, and then depositing the

without covering, however, patching is usually not advisable, as the cost of an additional application is small.

In patching an ordinary oil or tar surface, one should first attend to the depressions. Slight depressions can be filled with oil, or tar, and sand, gravel, or chips; but if they are deep, stone or slag should be added. The depressions should be cleaned, and the edges cut out if necessary, in order to allow a thorough bonding of the new and old material.

Although the best results are obtained by mixing the bituminous material and sand, or stone, etc., before applying, satisfactory results may be obtained without mixing, if the work is done carefully. In filling slight depressions, where the materials are not mixed before applying, a thin coating of oil or tar is spread over the bottom and covered with sand, gravel, or chips. If stone or slag is used with oil for filling deep depressions, the stone or slag is spread over the oil with which the bottom has been coated, then more oil is applied and covered with grit. If tar is used, the methods are similar, but more bituminous material is required.

After filling the depressions, any bare places, or places where the bituminous surface is practically worn through, should be coated, the surface first being cleaned and grit being spread over the oil or tar. In no case should much more than ½ gal. of bituminous material per sq. yd. be used for coating the surface, as any surplus oil or tar will result eventually in the rolling of the surface and the formation of bunches.

Most of the so-called cold tars may be applied without heating, as may the lighter grades of the so-called cold oils, or road oils; but the heavier grades must be heated in order to give the best results in patching.

A much heavier oil or tar than that used in the surface is inadvisable for patching, yet a somewhat lighter grade may be used. Excellent results are obtained by patching a hot oil or tar blanket with the heaviest cold oil or tar, and with these there is much less chance for the formation of bunches than with the heavier materials.

If a pavement composed of stone and bituminous material is constructed properly, not much in the way of maintenance should be required for some time, but depressions will develop eventually, and some patching will be needed. In patching bituminous pavements, including those composed of sand and oil, the methods are similar to those for a bituminous surface, except that, with the sand and oil pavements, deep depressions should be filled with heavy oil and sand, properly heated and mixed before applying; or, if the heavy oil is not available, or there are no facilities for heating the sand, an oil similar to that used for surface work will give good results with slag or cinders, without mixing.

Mr. Farrin ton

Covering.—On any bituminous surface, and especially one in the formation of which oil is used, more or less covering with sand, gravel, or chips will be required, either to take up free oil or tar, which will work to the surface for some time after it is applied, or later to prevent the picking up of the surface, which may soften under certain conditions. Covering is also partly a remedy for the slippery condition incidental to any bituminous surface during cold weather.

Re-Oiling or Re-Tarviating.—For re-treating a bituminous surface, practically the same equipment is required and the same methods should be followed as for the original treatment, except that usually less bituminous material is needed.

The life of a bituminous surface, or the length of time elapsing before re-treatment is necessary, will depend on local conditions, and the amount and kind of travel, etc. Where conditions are favorable for such treatment, however, and the travel, especially the horse-drawn travel, is not too heavy, a bituminous surface consisting of grit and about ½ gal. per sq. yd. of the heaviest hot oil suitable for blanket work should last from 3 to 5 years without re-treatment, if patched occasionally and otherwise maintained properly. In re-treating such a surface, not more than ¼ gal. of oil per sq. yd. should be used, unless practically all the original surface is gone; and it is not economical to delay re-treatment until this point is reached, as without the protection of the bituminous surface, the road metal will tend to ravel and the roadway to disintegrate.

A bituminous surface formed by the application of from ⅙ to ¼ gal. of light oil per sq. yd., with a small quantity of covering, will last at the best not more than one season, and it will probably be necessary to cover such a surface quite heavily in the fall in order to carry it through the winter and until another application is made the following year. Where from ⅜ to ½ gal. per sq. yd. of the heaviest cold oil is used with grit, re-treatment with about the same quantity of oil will usually be necessary the second year; but, after that, the application of from ⅙ to ⅓ gal. per sq. yd. per season should be sufficient, and an application may not be necessary each year, although, if omitted, the patching will naturally be increased.

If refined coal-gas tar is used, about ½ gal. per sq. yd. is required during the first and second seasons. After that, with cold tar, a ¼- to ⅓-gal. application will usually be necessary each season, although the yearly application may be omitted occasionally. With hot tar, ordinarily a ¼- to ½-gal. application should be made every second year. Where unrefined water-gas tar is applied, two ⅜-gal. applications are advisable each year; but, if only one ½-gal. application is made, a considerable quantity of patching will be required.

As to equipment, for handling the lighter grades of bituminous materials, one or two distributor carts and a pump for transferring

these materials from the tank-cars are required, and there should be a tool-house, also tents for the laborers, unless vans are provided.

For handling the heaviest grades of oil or tar used for surface work, the following plant is needed:

- One 10- or 12-ton steam roller,
- One portable heating boiler,
- One steam pump,
- Two steel tank-wagons,
- One pressure distributor (if mounted separately, or two if attached to the tank wagons),
- One or two watering carts,
- One 2-horse sweeper,
- One portable tool-house, and
- Two large tents or vans for laborers.

As to distributors, for handling the road oils or the corresponding grades of tars, a gravity machine may be used with good results, if the bituminous material is broomed after it is applied; but the cost of distribution will be less and the results more satisfactory if a pressure distributor is used. For this work, a distributor cart with a pump mounted on a frame back of the tank, and connected by chain with a large sprocket bolted to one of the rear wheels is most satisfactory. Under favorable conditions, good results have been obtained with gravity machines in distributing even the heaviest grades of bituminous materials for surface work; but the chances of failure, if the conditions are not favorable, are such that they should always be applied by pressure distributors if possible. The distributor just described, or one with a gas-engine pump mounted either on the tank-wagon or on separate gear, may be used, but one operated by steam will give the best results. One advantage of the steam distributor is the possibility of cleaning out the pipes and nozzles by blowing steam through them. Of the steam distributors, there seems to be little choice between the one using the direct pressure of steam in the tank for distributing and the pump type, each having its advantages.

To facilitate covering, the sand or other material should be piled in advance on the side of the road.

Before commencing to distribute the bituminous material, the surface should be cleaned with a horse sweeper, and with hand-brooms, if necessary. Any depressions should be filled and the surface patched as needed.

Where the heavier grades of oil or tar are used, it is well to water the old surface before distributing the new bituminous material, but watering is not necessary with the lighter grades unless the surface is unusually dry.

Mr. Farrington.

It is advisable to cover the bituminous material lightly at first, applying more grit as needed, for though some bituminous material will take up only enough grit to form a good mixture, without reference to the quantity put on, in other cases, if there is surplus covering, the oil or tar will continue to work up until the mixture is weakened to such an extent that the best results cannot be obtained.

Seal Coats.—Although a seal coat or wearing coat may not in all cases be necessary on pavements composed of broken stone and bituminous material, the writer believes that it is economical and should usually be applied, either in connection with construction, or later, except possibly where asphalt is used in the pavement. The equipment and methods for applying or renewing such coats are similar to those for the re-treatment of the surface; but, unless asphalt is used, the bituminous material for the seal coat should be of a lighter grade than that incorporated in the pavement. Oil may be used with good results on a tar pavement, and is advantageous, in some cases, in that it is less slippery than tar and also acts to some extent as a dust absorbent.

Re-Shaping or Re-Surfacing.—Although a surface consisting of grit and suitable bituminous material will carry ordinary travel and relieve the road metal of much of the wear, it alone is not a permanent form of treatment. As a result of the gradual wear of the road metal, and because repeated treatment and patching with bituminous material will cause the formation of bunches and an uneven surface, re-shaping or re-surfacing of the roadway will be necessary, eventually.

When this stage is reached it is always well to consider a more permanent form of pavement; for it is an established fact that a bituminous surface will not carry a large amount of horse-drawn travel, and will fail under certain conditions which the writer believes are not as yet clearly understood; and although a bituminous surface may have been satisfactory in a certain locality in the past, the conditions may change or the travel increase largely.

It is assumed, however, that there will not always be money available for the construction of a bituminous pavement, in which case the alternative is the re-shaping or re-surfacing of the roadway and the application of another bituminous surface.

The equipment needed for re-shaping or re-surfacing work, to be done in connection with re-oiling or re-tarviating, is the same as that for re-treatment, except that two steam rollers will be needed, also a straight-tooth and a spring-tooth harrow.

If stone is to be crushed, there should be a portable crushing outfit.

Mr. Harrington.

In re-shaping or re-surfacing a roadway with a bituminous surface, the old surface should first be thoroughly broken up with the roller picks or a scarifier and removed with stone forks.

The loosening of the road metal, partly accomplished by the roller picks or scarifier used in breaking up the surface, should be completed with a straight-tooth harrow, the use of which also tends to shake the worn out material to the bottom. This should be followed by a spring-tooth harrow, which, because of the curved teeth, will bring the stone or larger particles of road metal to the top. The surface should be shaped with rakes, any new stone or other road metal needed should be put on, and the road watered and thoroughly rolled. If any considerable quantity of new stone, etc., is used, the voids should be partly filled with sand, stone screenings, or chips; but, if only a small quantity of new road metal is put on, there will usually be sufficient fine material in the road to bond it.

Either a macadam binder or the heaviest road oil may be used, but, ordinarily, the former will give the best results. About ½ gal. of oil per sq. yd., should be used in two applications, with a covering of grit after each.

Ordinarily, it is not advisable to use tar for such re-shaping or re-surfacing work, unless a bituminous pavement is to be formed by penetration. In this case, usually more new stone or other road metal is needed, and the voids for about 2 in. below the surface should not be filled with fine material. For grouting 1¼ gal. of heavy tar should be used, and a seal coat should be formed by the application of about ½ gal. of a lighter tar with grit; or, if preferred, asphalt may be used instead of tar.

The foregoing description of maintenance work applies to a bituminous surface on a roadway of broken stone, slag, or screened gravel, rather than to such a surface on unscreened gravel; but though the methods for handling a surface on the latter form of roadway are somewhat different, mention can only be made of the fact that although, for the first application, one should use a fairly light grade of bituminous material which will penetrate somewhat below the road surface and thus form a real bond between the bituminous material and the gravel, heavier grades of oil or tar may be used, with good results, for later treatment.

Re-Surfacing of Bituminous Pavement.—In a discussion on maintenance, it does not seem advisable to devote much space to the re-surfacing of bituminous pavements, because such work is probably better classified as reconstruction than maintenance, but attention is called to the fact that the life of pavements composed of broken stone and bituminous material is as yet problematical on roads outside of cities

and large centers, because such pavements have been in use, under these conditions, only a few years. Mr. Farrington.

Storage and Repairs.—Incidental to a consideration of equipment is the question of proper care and up-keep of the outfits. There should be at headquarters, a suitable storage place, with sufficient equipment and proper facilities for making ordinary repairs. It is not only advantageous to have the repairs made by or under the supervision of those responsible for the apparatus during the working season, but, by having the work done by men from the regular force, it is possible to keep the most competent men busy during the winter.

If the amount of work done warrants the employment of a storekeeper, there should be kept at all times, at the storage place, a sufficient stock of tools, etc., to supply the needs of all the gangs.

Reports and Accounts.—It is not possible, in the time at the writer's disposal, to consider in detail the question of the reports, records, and accounts which should be kept on maintenance work. The writer believes, however, that although a consideration of cost is not of especial value in a discussion covering work in widely separated localities, a comparison of the cost of work in different localities in one section is valuable, and is necessary in order to determine what equipment and methods are most economical, and also whether the work is handled efficiently.

Daily reports giving details of the labor cost should be submitted by the foreman, and a record of these, as well as of bills for materials, supplies, etc., should be kept in such a way that the cost of the different kinds of work can be easily ascertained at any time.

H. B. Pullar, Assoc. M. Am. Soc. C. E. (by letter).—The methods of maintaining bituminous surfaces and pavements are unquestionably of great importance. Mr. Farrington has covered very fully the matter of maintaining bituminous surfaces and bituminous roads constructed by the penetration method. He has stated that he does not consider it advisable to devote much time to the re-surfacing of bituminous pavements. Although it may not be advisable to devote much time to the total re-surfacing of bituminous pavements, much more might be said regarding their maintenance and repair. The method and equipment necessary for repairing sheet-asphalt and asphaltic-concrete pavements in small towns and cities have been of great importance, and it is on account of the fear of the cost and trouble in making satisfactory repairs that many small towns have given preference to brick, block, and other types in place of bituminous pavements. If proper care is taken and proper equipment is secured, there is no more trouble, nor any greater expense, in repairing bituminous pavements satisfactorily than with Mr. Pullar.

Mr. Pullar. any other kind. Methods and equipment necessary for repairing these pavements will be discussed later.

It has only been within recent years that much consideration has been given to the repairing of sheet-asphalt pavements. Until recently, the only thing which could be done was to cut out the bad part of the pavement and replace it with new. On streets where the total repairs are more than 50 or 60% of the pavement, this is still the best method. Where this plan is used, all the bad material should be cut out, the edges of the old pavement should be carefully painted with the pure bituminous cement, and then the new material should be put in by a method similar to that used in constructing the pavement. As Mr. Farrington states, an effort should be made to make the repairs with materials of the same class as those used in the original pavement. In the larger cities, or wherever it is possible, the most economical way to obtain the mixture is from some paving contractor. In smaller cities it is advisable to obtain one of the small mixing plants which are on the market and now available. The "Rapid" mixer has proved to be of exceptional value for this work, and is now used by a number of cities in the Middle West for repairing both sheet-asphalt and asphaltic-concrete pavements.

For sheet-asphalt pavements, in which only small depressions or waves appear, the most economical and probably the best method is to burn off about ½ in. of the top and make the repairs by what is known as a "skin patch". This method has been used very successfully in various parts of the country, and there are a number of excellent burners on the market. The burning can be done at about the same expense as would be involved in ripping up the pavement, and in this case it is only necessary to put on a patch about ½ in. thick, instead of cutting out and replacing the entire thickness of the pavement.

The "Lutz" heater method for repairing is based on the same principle—burning off a small quantity of the top surface and replacing it with a "skin patch". The surface is heated by a hot-air blast, and claims are made that no injury is done to the pavement, with the exception of the upper ½ in. which is removed. This heater has been used successfully by a number of the larger cities for re-surfacing work, and it is possible to repair about 500 sq. yd. per day with one machine. Care and experience, however, are necessary to repair or re-surface pavements properly by burning. It should not be attempted by anybody without some knowledge of the proper method. It is not a method which can be recommended for general use in small cities or towns, or on public highways.

During the past few years many attempts have been made to re-melt and use the old material for repair work, but, up to the present time, no successful method has been developed. The Noyes crushing machine has been used successfully for pulverizing the old material.

but the reheating has proved a difficult matter, and thus far has not met with success. The material, after being pulverized by the Noyes crusher, can be used to a considerable extent in a binder mixture, and this idea is being adopted in a number of places. Some of the small mixers claim to be able to re-melt old material satisfactorily, but in nearly every instance considerable burning takes place. Mr. Pullar.

About 2 years ago the writer made a new experiment in repairing sheet-asphalt pavements. The section of pavement to be repaired was very badly cracked, and contained three holes, each about 1 sq. yd. in area. The holes were first cleaned out and the edges trimmed. Stone, ranging from ¼ to 1 in. in diameter, was then placed in each hole and tamped, and to this there was applied about 1 gal. per sq. yd. of a heavy asphalt binder. Chips were then spread over the surface and thoroughly tamped into the voids. Then a squeegee or seal coat was applied over the entire surface, including the stone patches. A recent inspection of this small strip of pavement showed that it was still in excellent condition, so that, without question, this method has already prolonged the life of that particular piece of pavement 2 years, and from indications, will continue to prolong its life another 2 or 3 years. On account of the success of this experiment this method has been used for repairing sheet-asphalt pavement in two or three other small cities in various parts of the country, and up to the present time, reports have been very favorable. This method could be used very satisfactorily in small towns and villages, and would be much more economical than the use of brick or cement for repairing bituminous surfaces.

In repairing asphaltic-concrete pavements of various types, the best method is to use a small mixing plant, such as the "Rapid" mixer, or other small mixer of similar construction. The nominal cost of these machines and the ease with which repairs can be made, will probably result in their more general use for repair and maintenance work.

There are two principal causes for trouble with bituminous surfaces consisting of the superficial treatment by use of tar, oil, or asphalt. One is the fact that, in a number of instances, too much bituminous material has been used. This produces a pavement which is very apt to rut or become wavy, and it is also very disagreeable to the eye. The other source of trouble is due to lack of sufficient bituminous material, or too great a length of time intervening between treatments. If the surface is carefully inspected at frequent intervals, and treated at the proper time and with the proper quantity of material, the road or pavement can be kept in excellent condition at a small cost.

The writer agrees with Mr. Farrington in the statement that it is not advisable at present to discuss the cost of maintenance, as this

Mr. ullar. varies considerably in different parts of the country, and, to a great extent, depends on local conditions. Cost data would be valuable even when work in different localities is considered, provided there was a standard system for giving cost, and one which would take care of different units. The total cost per square yard is of little importance unless unit costs are given. A standardization of a system for giving cost data, in both paving and road construction, and in paving and road maintenance, is greatly needed, and much valuable work could be done by a committee of engineers appointed for this purpose.

The question of equipment and methods for maintaining bituminous surfaces and bituminous pavements, is a very broad one, and it is impossible to cover it fully in one discussion, but it is hoped that further experience may be cited on this same topic, especially in reference to a cost system and to the best methods for use in small towns and villages.

r. hard. ARTHUR H. BLANCHARD, M. AM. SOC. C. E.—Mr. Farrington properly considers the work of maintaining bituminous surfaces and bituminous pavements under three heads: first, routine maintenance, being the continuous repair of small areas of a surface or pavement; second, re-applications of bituminous materials over practically the whole surface; third, reconstruction of the wearing course.

The second and third classes of maintenance may be accomplished by day labor or contract, or a combination thereof, the problems involved being usually identical with those of primary construction. The relative advantages of the general methods referred to and the plant equipment required have formed the subject-matter of several previous discussions before this Society.* The speaker will confine his discussion to the field of maintenance work first mentioned above.

In considering the equipment required to accomplish continuous maintenance of bituminous surfaces and bituminous pavements, it seems desirable to cite certain factors and principles which appear to be essentials of economical and satisfactory maintenance.

1.—Small failures of the wearing course, and the wearing away of the bituminous surface on comparatively small areas should be repaired immediately. Otherwise the rate of disintegration will be materially increased, the annual maintenance charge will be greater than necessary, and the surface will be unsatisfactory to the users of the highway.

2.—Continuous maintenance should be conducted so that the highway may be at once opened to traffic without injury resulting to the repaired sections. For the repair of potholes, ruts, and other holes or depressions, this implies the use of bituminous materials of much lower penetration than have ordinarily been used in many cases. If

* *Transactions*, Am. Soc. C. E., Vol. LXXIII, p. 25; Vol. LXXV, p. 548; and Vol. LXXVII, p. 171.

practicable, the same type and grade of bituminous material should be used for the surface application as was placed in the original construction. Mr. Blancha

3.—In order to accomplish satisfactory results the labor should be efficient and should be under the supervision of an engineer experienced in the construction and maintenance of all types of highways, and with all kinds of bituminous materials used in the area under his jurisdiction.

4.—The intimate relationship between the methods and materials used in construction and the character of the maintenance, in the case of bituminous surfaces and bituminous pavements, is such that, to ensure success, the construction and maintenance of highways in a unit of area, or a certain mileage, should be under the supervision of one engineer.

To accomplish continuous maintenance economically and efficiently under the foregoing conditions, the flying squadron, operated under the direction of an engineer in charge of construction and maintenance of highways in the area covered, appears to be the logical solution. The equipment of such a squadron will naturally depend on local conditions, such as the mileage of highways to be maintained, their relative location, the types of surfaces and pavements, the kinds of bituminous materials used, the mileage under guaranty, etc.

As an example of a definite problem in continuous maintenance, take a county or a division of a State where a considerable mileage of bituminous surfaces and bituminous pavements has been constructed. Granted a well-maintained system, the work to be accomplished by a flying squadron would consist, first, of routine repairs, including filling all holes, ruts, and other depressions with bituminous concrete, using a type of aggregate and bituminous material suitable to each case, and, second, applying bituminous materials to all areas which gave indication of being in such condition that they should be re-treated at once rather than wait until the whole surface required another application. It will be practicable in many cases for the flying squadron to repaint guard-rails and perform other routine repair work. Under such conditions, it is believed that the use of a motor truck properly equipped will prove most satisfactory. The suggestion to use a motor truck in repair work on highways is not novel, trucks having been recommended and used for this purpose for several years.

The equipment to be carried by the motor truck for the special repair work outlined should include the following machinery and tools: a rotary heater and pug mill mixer; two heating tanks; a surface heater; storage tanks and barrels for bituminous materials of different types and grades; storage capacity for small tools such as brushes, squeegees, tampers, cutters, pouring cans, irons, shovels, picks, and hoes; and storage capacity for paints for guard-rails, mineral aggregates, and road metal of several sizes. Although a storage capacity

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chard.
for mineral aggregates and road metal is called for in this equipment, it should be noted that only a few cubic feet of these materials will be carried, because the maintenance work performed will be confined to the repair of small patches. Usually, it will be practicable to reload a truck each day. The fixtures for the equipment described should be arranged so as to be readily removed, thus allowing the truck to be utilized for general hauling purposes. The equipment recommended should not be confounded with the elaborate equipment designed to cover all phases of repair work recommended by some engineers. The speaker believes there is danger in overloading a truck with accessories and thus rendering its operation uneconomical.

The equipment recommended herein could be carried by a 5-ton truck, capable of traveling 12 miles per hour, and having a gasoline storage capacity for a run of 100 miles. A clear body of 7 by 15 ft. would be available for the installation of the equipment. The cost of operation during an 8-hour day would vary from $10 to $20, covering wages of chauffeur, rent of garage, interest on first cost, maintenance charges, depreciation, insurance, tires, gasoline, oil, and grease. The speaker is indebted to Mr. Alfred F. Masury, Service Manager, The International Motor Company, for details relative to truck operation.

Ir.
veiler.
W. H. Fulweiler, Assoc. M. Am. Soc. C. E. (by letter).—It is not economical to apply a bituminous material without some covering, as the greatest value of the treatment lies in the added wear given to the surface of the road by the mineral aggregate used as a covering, which the bituminous material merely serves to protect and retain in position. It is always economical, especially in the re-treatment of bituminous surfaces, to repair the surface before applying the bituminous treatment.

In patching, it is very important to see that the edge of the patch is cut back slightly, so that it not only forms a square edge for the new material, but that the material on the edge of the patch, which is invariably partly disintegrated by the action of the water, is removed.

The writer would like to inquire whether Mr. Farrington has any explanation for his statement that "Where tar is used in patching, more of it is required than with other materials."

As to the quantity of material to be applied on the surface, the writer's experience indicates that should it be desired to apply $\frac{1}{2}$ gal. as a re-treatment on a bituminous surface, the best results would be obtained by applying not more than $\frac{1}{4}$ gal. at a time—with just sufficient covering to enable the wagon or motor truck to pass over the surface—then applying the remainder of the material and adding the stone used for covering. Where such a large quantity of material ($\frac{1}{2}$ gal.) is used in re-treatment work, it is advantageous to use stone

of a larger size, in fact the writer's best results have been obtained with a size commercially known as "$\frac{3}{4}$-in." Mr. Fulwe

The writer agrees with Mr. Farrington regarding the excellent results that may be obtained in patching with bituminous materials which are prepared so that they may be applied without heating, as this removes one of the serious difficulties in making a large number of small patches. Furthermore, such materials may be applied at almost any season of the year, and are much less affected by the presence of dust and moisture in the road surface.

In this day of definition and striving to attain accuracy in the use of technical phrases, the writer would like to inquire whether Mr. Farrington believes it to be good form to use a proprietary trade name in the way he has used the terms "Tarviating" and "Re-tarviating"?

In reference to the use of tar in re-treatments, the writer's experience has been that where the road is in good condition and an initial application of $\frac{1}{2}$ gal. per sq. yd. is made, that under favorable conditions, patching is all that is necessary during the second season, and about $\frac{1}{3}$ gal. per sq. yd. for the third season, followed by $\frac{1}{4}$ gal.; although, under many conditions, $\frac{1}{3}$ gal. every other season seems to be sufficient.

The writer does not agree with Mr. Farrington in reference to applying the covering to the bituminous material lightly at first. After a little experience it is very easy to estimate the proper quantity, and better results are obtained by putting on this correct quantity immediately. This saves trouble, annoyance to traffic, and prevents the formation of spots in the road, where, owing to the small quantity of covering, the road surface will pick up, especially under heavy steel-tired traffic. It is but fair to state that the writer is drawing purely on his own experience, which has been confined practically to a single type of material.

Slipperiness frequently results from the method of putting on the stone covering, as suggested by Mr. Farrington, as the application in small quantities tends to keep an excessive quantity of bitumen in the top of the road.

In connection with the description of the organization for surface treatment and re-treatment, the writer would be very glad to have an expression of opinion from Mr. Farrington as to the number of square yards per day treated by such organizations, assuming an average haul of 3 miles from the railroad station.

Mr. Farrington mentions a macadam binder, or the heaviest road oil to be used in re-shaping the surface of an old road. By macadam binder, does he mean a particular grade of bituminous material, or the usual stone screenings well watered and rolled?

In reference to maintenance work on screened gravel, the writer is in entire agreement with Mr. Farrington, that a fairly light grade

r. iler. of bituminous material should be used on the first application, followed by a heavier grade, but finds it best to apply the heavier as soon as the light material has been absorbed into the surface. This same method has been used successfully on oyster-shell roads.

In re-surfacing old bituminous pavements, the writer's instinct would always be to advise scarifying, but the results of some work done during the past season would seem to indicate that, with care, it is possible to secure very good results by using crusher-run stone, free from dust, that will pass a 1½-in. screen, applying this as a covering over about ½ gal. of material, rolling thoroughly, applying about ⅓ gal. and covering with stone chips.

In the organization for re-surfacing, where a considerable amount of this work is to be done, the writer would like to call attention to a plan by which a very large amount of such work has been done on the State roads in Maryland during the past several years. Under this plan the State furnishes and applies the necessary stone covering, purchasing the stone and having it delivered in piles along the roadside in the early spring, when the price of teams is quite low. From a schedule prepared by the State engineers, showing the location of the roads to be treated, a price is agreed on, and a contract drawn up whereby the manufacturer of the material agrees, for a unit price per square yard, to sweep surfaces which have been previously treated with bituminous material, and also surfaces which have not been thus treated, and a unit price per gallon for applying the material, at the approximate rate shown on the schedule. In this contract for the past season, it was specified that all work should be completed before July 31st, with a penalty clause of $25 per day for every day that this time was exceeded. There was a further clause in which the company agreed to pay the State $25 per day for every day that it was delayed through breakdowns or the failure of material to arrive, and the State agreed to pay the company the same amount for every day that it delayed the application of the material through inability to cover it. This contract involved some thirty-four pieces of road in nine counties of the eastern shore of Maryland, amounting to about 118 miles, and containing 976 071 sq. yd. On this was applied 342 785 gal. of material, at a rate varying from ⅕ to ½ gal. per sq. yd., and averaging 0.366 gal. per sq. yd. This was covered with stone chips averaging 12 lb. per sq. yd.

Two motor trucks were used in this work, and they arrived about May 26th, although, owing to bad weather and some delay in organizing the gangs, work was not commenced until May 28th. One truck finished practically half of the work on July 18th, and the other finished on July 30th. The State maintained two gangs, of 12 men each, each under an inspector, for applying the stone chips, the company sweeping the roads and furnishing and applying the binder.

Practically this same form of contract had been in force during 1912, but during that year the work was not laid out according to schedule, and the trucks frequently had to make long jumps between the different pieces of work. This caused considerable loss of time, which was furthermore augmented by difficulty in securing proper labor. As the result of the experience of 1912, the work was laid out more systematically in 1913, and men composing the covering gang in one case were held throughout the season. The effect of having experienced men for covering is well brought out in Table 1 which summarizes the operations of the two years. Mr. Fulweil

TABLE 1.—Summary of Operations for 1912 and 1913.

	1912.	1913.
Sunday and holidays	21	20
Repairs and delays	22	6
Rain	5	9
Transit	34	18
Applying material	57	72
Total elapsed days	138	125
Percentage of days worked	66%	72%
Percentage of days material applied	41.5%	57.5%
Average No. of gallons applied per day	2 860	4 760
Average No. of square yards treated per day	8 230	13 550

The time is given in truck-days.

In 1913 the decrease in time lost in transit, repairs, and delays, is quite striking, as is the average increase in the quantity applied per day.

The writer believes that, in the long run, this plan, where there is a considerable mileage to be treated, is probably the most efficient and economical method of handling this maintenance work. One point, which is of considerable importance, is that, contrary to the case of an ordinary contractor, the company is vitally interested in securing the very best possible results from the use of its material, and therefore the inspector's duties are very much lightened, as there is no incentive to skimp the sweeping or other details in the application and then blame the quality of the material for the poor results which would inevitably ensue.

James H. Sturdevant, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Farrington has well covered the subject of maintenance, without which any road system becomes an abomination instead of an improvement. Many times the writer has listened to tourists recounting a day's experience on the roads. They have expressed great indignation concerning some rough spots, but have not mentioned the many miles of smooth, easy-riding roads over which they have passed. Mr. Sturdevant.

There are a few minor points to which the writer would like to call attention. These may seem to be trivial, but, if attended to, will result in ultimate satisfaction to the general public.

Mr. Sturdevant.

Patching.—In order to keep the surface of a road in smooth, proper condition, it should be under constant surveillance, and receive constant attention. As the surface becomes worn in places, and holes and ruts develop, they should be repaired at once, for if neglected they rapidly extend in size and depth under the action of each traveling vehicle. On bituminous surfaces, the best results are obtained, as stated by Mr. Farrington, if the asphaltic materials are first mixed with stone, gravel, or other suitable ingredients. This mixing can be done at any time, and in sufficient quantities to last for several months, for the weather will not cause the mixed material to deteriorate. The writer has used material which had been mixed during the previous summer and piled by the roadside, and has obtained as good results as with fresh materials.

In all patching work, the first requisite is to see that the surface to be repaired is well cleaned before new material is placed; also, in repairing holes and ruts, care must be taken not to use too much asphaltic material, and it is well not to have the new patch flush with the surface, for, under the action of the sun, the asphalt, whether light or heavy, will come to the surface. Then the patch will become sticky and is likely to be pulled off by heavy slow-going vehicles. To prevent this, more stone and fine material must be used as covering, and, if too much was used originally, a hump will develop which is as disagreeable as the original rut or hole. This is not as likely to occur when the material has been mixed before placing, as when the asphalt is placed first and then covered. Where the patching has been properly done—judging from the writer's experience and observation—it will last long after the original oil and surface have disappeared. This is true as to either light or heavy oils, or even asphaltic binders.

Re-oiling.—For re-treating bituminous pavements, the quantity of material to be used varies with individual roads. The original treatment should be in sufficient quantities to bind the road thoroughly and hold the stone in place, to prevent raveling; subsequent applications are for the purpose of providing a mat or cushion. For this purpose, the lighter oils are generally used, though excellent results, well worth the extra cost, are obtained with heavy binders. Considering, however, the lighter grades of oil, it is apparent that if used in excess they are not heavy enough to sustain the weight of traffic, and a rutted condition is sure to result. To avoid this, the application of oil should be uniform and of just sufficient depth to cover the surface, with no free oil remaining. After being covered, the surface should be watched closely, especially in warm weather, and as soon as the oil appears through the surface, more material should be added. In general with all bituminous surfaces, satisfactory results can only be obtained by keeping close watch of the road and repairing all defects as rapidly as they appear.

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PAPERS AND DISCUSSIONS

THE EFFECT OF SATURATION ON THE STRENGTH OF CONCRETE.

Discussion.*

By J. L. Van Ornum, M. Am. Soc. C. E.†

J. L. Van Ornum, M. Am. Soc. C. E. (by letter).—It is gratifying to note the several instances cited as showing the marked change in strength which results from saturating concrete which has been previously exposed in air. Undoubtedly there are many others, that have not been reported, which would indicate the same general tendency. Of course, the proportionate loss of strength occurring when an air-cured concrete is saturated would depend on a multiplicity of existing conditions, some of which have been referred to. One of these is the relatively large area of surface compared to volume in the test specimens, as noted by Mr. Worcester. It would seem probable, however, that the difference in loss of strength between the concrete in structures and that of the experiments would not be nearly as great as suggested by those comparative differences, because there is the partly compensating fact that structural members are generally exposed during construction to the drying effect of breezes and often to direct sunlight and a dry air, while the specimens were in a small closed room in which the air was comparatively moist. If, on the contrary, "the surface were kept wet for 7 days," it is believed that a large part of the loss on saturation would be avoided. Mr. Van Ornum

With regard to the values of deformations and sets, which Professor Hatt assumes were withheld, it can only be stated that circumstances prevented their determination in connection with the tests reported. The only additional information pertinent to the subject, which can

* Continued from January, 1914, *Proceedings*.

† Author's closure.

Mr. Van rnum. be added, has reference to the suggestion of Mr. Worcester, that the actual strength of the specimens cured entirely in air "must have been rather lower than would naturally be expected of this quality of concrete." These cylindrical prisms averaged 1 760 lb. per sq. in. Though this is about one-tenth less than the compressive strength that may be fairly expected of good laboratory specimens of these materials and proportions, they were actually somewhat stronger than the average of a large number of test prisms of similar concrete used in building construction and supposedly treated during curing in a way corresponding to that given to the material in the structures.

The writer hesitated to publish the paper at all, because of the many supplementary questions remaining for investigation, the determination of which is necessary for a complete understanding of the phenomena involved. Nevertheless, he concluded that the field of further research would be made more definite and available to others, if he presented the preliminary results already given. Of course, other percentage values will result from different experimental details, and the curve itself may be varied considerably in the same way. Yet it is believed that the broad, general fact of a significant loss in the strength of dry concrete when suddenly saturated and its gradual increase in strength to a final value considerably greater than the original one, when the saturation is prolonged, is a reality which should be definitely recognized in dealing with this important engineering material.

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PAPERS AND DISCUSSIONS

MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

By G. E. P. Smith, M. Am. Soc. C. E.

G. E. P. Smith, M. Am. Soc. C. E. (by letter).—The author's discussion of weirs is particularly timely, inasmuch as the necessity for the measurement of water by all water users is being felt at the present time. There are comparatively few irrigation projects in the Southwest where the water is not now delivered, nominally at least, by measure. Mr. Smith

The author has rendered valuable service in collating the various masses of experimental data on the flow over suppressed weirs. There appears to be a general agreement among experimenters within 3%, though a divergence of results as high as 5% is not infrequent. Apparently, there is need of another set of experiments in which conditions and methods are defined much more rigorously than any thus far attempted. If an important new set is made in the future, doubtless the author will wish to amend his formulas. On account of the nature of the problem, no formulas can be accepted at the present time as final.

The Francis formula, a sort of average law, has been disintegrated into a multitude of formulas, each covering a limited range of conditions. This is in the interests of accuracy, and here the author has succeeded very well, but the writer cannot agree with him that it is in the interests of simplicity. Of course, the general use of accurately platted diagrams avoids the use of cumbersome tables, but errors in taking results from the diagrams can easily exceed the discrepancy between the author's and the Francis formulas.

* Continued from January, 1914, *Proceedings*.

Ir. ith.

One assumption made at the outset of the paper is perhaps open to question. It is that the discharge is proportional to the length of crest. The influence of the side-walls on the discharge must be variable, depending on the material of which the walls are built and the character of the workmanship. The influence is comparatively slight for long crests, but of considerable importance in the case of short ones, such as are found in irrigation laterals and head-ditches. A serious difficulty in applying the author's method is the standardization of the side-walls. It is much simpler to standardize an edge than a wall.

It would seem to be unnecessary, too, to place the gauge so far up stream as is recommended. The object being to get back of the "surface curve," which depends mainly on the head, it is more reasonable to place the gauge at a distance back dependent on the head under ordinary or average conditions of the stream, say, at a distance equal to four times the head.

The writer formerly used the standard rectangular weir, but abandoned it gradually in favor of the Cippoletti weir, on account of the greater ease and simplicity of installing the latter. The Cippoletti weir requires much less construction than a suppressed weir, a pool being formed up stream in place of a flume. On one occasion the writer built and set an 18-in. weir, got a satisfactory reading and caught a train in 80 min. The Cippoletti weir discharge is proportional to the length, and a table of discharges is so simple that an engineer is not required to interpret it. Probably the greatest error in its use is in neglecting the velocity of approach, and this error can be minimized by providing an ample height of weir, and by applying the usual Francis correction. Furthermore, it would be possible to analyze the Cippoletti weir in a manner similar to the author's, and the subsequent use of the "approved" tables would result in the highest degree of accuracy possible with weirs.

Whatever doubt the writer had regarding the accuracy of the Cippoletti weir was set at rest by observing an exceedingly valuable set of experiments in the hydraulic laboratory of the University of Wisconsin. These tests were performed by senior students under the direction of Professor G. J. Davis. They indicated that:

1. The ratio, 1:4, is correct for the side slopes;
2. The coefficient, 3.37, is correct for crests more than 6 in. long;
3. The error of the formula will be less than 1%, if the head does not exceed 0.4 of the length of crest.

The restriction sometimes placed on the rectangular weir is that the head shall not exceed one-third the length of crest.

Mr. Smith.

There is one idea expressed in the paper to which exception should be taken. The proposition to establish one type of weir for universal use by legislation is debatable at least. Legislators are not engineers, and the fewer legal restraints thrown around the Profession the better it will be for the Profession. There are no laws fixing the power of the transit telescope, or the size of sewer pipe, or steel rail sections, nor are such laws wanted. The standard unit of measure should be fixed, as it is, but the method, never. Better methods will be found in the future, and should not be debarred from use in advance. There are several good ways to measure land, to measure electric current, and to measure water. The best method for a particular case depends on the surrounding conditions.

Utah has a law establishing standard cross-sections for the roads of the State. Is it possible that the southern counties and the northern counties require the same road crowning and the same shape of gutters? In Arizona roads are built variously of volcanic cinders, gravel, out-wash deposits, caliche, and sand; some roads are used almost wholly by automobiles, others by burros; some are in pine forests, others where the rainfall averages less than 3 in. per year; some withstand traffic principally, others rain and frost, others terrific winds. Surely no standard cross-sections are applicable here. Logan Waller Page, M. Am. Soc. C. E., states that "individual judgment is necessary" to determine the cross-sections of roads.

Instead of more standardization, perhaps more originality is needed. Engineers sometimes follow the well-beaten track when much could be gained by taking the cut-off—a statement which is worthy of the following illustration. For several years the writer had charge of a weir in a mountain canyon near Tucson. The discharge of the canyon varies from a small trickle in the dry months of the drier years to occasional floods of more than 1 000 sec-ft. It was quite as essential, perhaps more so, to determine the discharge of the years of minimum run-off as that of the infrequent maximum years. What was needed then, was an adjustable length of crest. Another consideration was the shape of the canyon; there were vertical rocks on the north side, at the foot of which ran the small flows, and a rising bouldery stream-bed from there to the south side. The weir crest, 65 ft. long, was designed to fit the canyon. For 5 ft. on the north side it was level, and the remaining 60 ft. had a slope upward of 1 in 15. Consequently, the small flows were concentrated over the short-level portion. The weir was built for one-third of what it would have cost with a crest of standard shape. The discharge formula for the weir is derived by using Francis' formula as a basis, and integrating separately for the triangular portion. The differential equation for the latter is

$$d\,Q = 3.33\left(\frac{x}{15}\right)^{\frac{3}{2}} dx$$

Mr. Smith. and the discharge for the entire cross-section is given by the equation,

$$Q = 16\tfrac{2}{3}\ H^{\frac{3}{2}} + 20\ H^{\frac{5}{2}}.$$

The equation was tabluated and also platted as a curve. Generally, the curve was used in working up the nilometer records.

There were errors, of course, due to neglect of the velocity of approach, and, probably, to submergence during extremely large floods. The influence of the former could have been approximated, but it was desired to have any possible error on the safe side. The action of the flow was such as to keep the up-stream side scoured out to a depth of from 1 to 2 ft. Records were kept for 9 years, during which time the nilometer box was submerged three times. The credit for the design of the weir is due to Professor S. M. Woodward, of the University of Iowa.

The writer, while traveling in the irrigated portion of Italy, saw many different methods of measuring water, among them various forms of weirs, submerged weirs, submerged orifices, and partly submerged orifices. It was interesting to observe that, in most cases, the method used was best fitted to the conditions of fall, discharge, accuracy desirable, and relative cost.

In the western part of the United States it is likely that a modification of the Venturi water meter will come into use to a considerable extent.

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PAPERS AND DISCUSSIONS

STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

By F. B. Marsh, Assoc. M. Am. Soc. C. E.

F. B. Marsh, Assoc. M. Am. Soc. C. E. (by letter).—This paper bids fair to be the starting point for much future consideration of this hitherto intangible question of the proper storage to be provided in municipal impounding reservoirs. For this reason the writer wishes to call attention to a factor which, although of relatively minor importance, yet becomes worthy of consideration when a water-shed has been developed well toward the economic limit, as, for example, the Croton. Mr. Marsh.

In allowing for evaporation in a well-developed water-shed, the area of water surface should not strictly be taken as that with all the storage full, because such a condition obtains generally for only a few weeks during the year, and in many years not at all. Moreover, the tendency is for it to occur at a season when the evaporation from water surfaces is small. The maximum evaporation occurs during the summer, when, as a rule, the storage has been depleted and the water surface is, therefore, smaller.

Opinions will probably differ as to what should be taken as an average condition of the reservoirs, but for present Croton conditions, at least, it is probable that the water surface corresponding to a condition of reservoirs two-thirds full would be a fair assumption. Certainly it should not be less than one-half.

The fact that the area of water surface is a continually variable quantity has led the writer to prefer stating catchment areas in terms

* Continued from January, 1914, *Proceedings*.

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of the total area, land plus water, rather than land area only, as used by the author. The correction for evaporation can apparently be made as readily on one basis as the other, and it has always seemed to the writer more convenient and logical to use the gross area, and less likely to confuse those who may have occasion to make practical use of the figures.

The author's warning that actual records of an individual stream, taken without reference to other data, are likely to be seriously misleading, even if they cover a considerable period, is strongly exemplified by the Croton record, where the average flow for the 18 years from 1869 to 1886 was 346 000 000 gal. daily, while for the next 18 years, 1887 to 1904, it was 449 000 000 gal. daily.

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PAPERS AND DISCUSSIONS

PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.

Discussion.*

By Messrs. H. A. Gardner, A. W. Carpenter, Maximilian Toch, R. D. Coombs, Lewis D. Rights, and W. E. Belcher.

H. A. Gardner,† Esq.—In referring to the behavior of various pigment paints, Mr. Sabin states that "our knowledge of paints is as yet largely empirical." This may be true in so far as it concerns any attempt to promote the use of a lead pigment, which is highly unsuited for the protection of structural steel, but it is a matter of great pride to the paint industry of the United States to point out the developments in paint technology which have been recorded during the past 10 years. In probably no other industry have such rapid strides been made. The achievements of Committee D-1 of the American Society for Testing Materials are especially noteworthy. The studies and researches of its members are largely responsible for the present state of this industry, in which the manufacturing processes are now controlled by the application of scientific principles. This change has led to a general abandonment of the slap-dash, secret-process methods of mixing paint, which at one time existed. Mr. Gardner.

Regardless of the progress that has been brought about through research and practical test, it is not uncommon to hear the phrase, "some thirty years ago", repeatedly used by the skeptic in referring to the old-time methods and materials of painting. To assert that there is but one paint or pigment that can be used for painting any and all kinds of structures, is like asserting that one brand of patent medicine should be used for the cure of any and all ills. In these modern days, it is the practice to design paints so that they will

* Continued from January, 1914, *Proceedings*.

† Assistant Director, The Inst. of Industrial Research, Washington, D. C.

r. dner.

properly meet special requirements. In no other way can the best results be obtained.

Unfortunately, Mr. Sabin's statements regarding the Havre de Grace Bridge tests are incorrect, and therefore misleading. In referring to the excellent results obtained with red lead paints, he states that the red lead used was 98% pure. The analyses made by the Department of Agriculture, at Washington, show that the red lead paints which gave good results varied widely in their composition. No. 6 paint consisted of 65% of red lead, the remainder of the pigment being made up of silica and other pigments. No. 10 contained 88% of true red lead and 10% of litharge. No. 11 contained approximately 95% of true red lead, the remainder consisting of litharge. Moreover, these three paints (the prime coats) were all made with different grades of oil and in different quantities. The oil in No. 6 which was present to the extent of 45%, had an ash content of 10.6, an iodine number of 140, and an acid value of 30. The oil in No. 10 was present to the extent of 30%, had an ash of 10, an iodine value of 157, and an acid value of 22. The oil in No. 11 was present to the extent of 23%, had an ash of 2.5, an iodine value of 163, and an acid number of 14. Paints 12, 14, and 16, which also gave excellent results in these tests when used at the normal spreading rate of 600 sq. ft. per gal., were composed of mixtures of red lead with such pigments as zinc oxide, China clay, gypsum, silica, asbestine, and carbon black.

It is hard to understand how any one can draw conclusions, from such a test, as to the protective value of different single pigments. The Havre de Grace Bridge tests have served a most excellent purpose in determining the relative value of various commercial brands of metal paints. These tests, however, cannot be cited as having contributed any definite knowledge relating to the effect of single-pigment paints as accelerators or preventives of corrosion. In fact, these tests were not designed for that purpose.

The only authoritative paint tests, to determine the efficiency of single pigments as preservers of iron and steel, which have ever been made, are those by the American Society for Testing Materials, at Atlantic City, N. J., in 1908. In these tests, which were made on a very extensive scale, the so-called rust inhibitive pigments proved the most satisfactory and received the highest ratings at every official inspection. Such pigments were either of the basic or the chromate type. The slightly soluble chromate pigments, such as zinc chromate, as Cushman had predicted, gave marked protection to the metal plates. The highly basic pigments, such as basic sulphate of lead, also gave excellent protection to the iron plates, to which they were applied as oil paints. Some of the most startling results of the test were recorded on test panels Nos. 34 and 36, and Nos. 9 and 10. Test panels No. 36 were coated with neutral chromate of lead. This pigment

is neutral and highly insoluble. It gave, therefore, only fair results. Test panels No. 34 were painted with basic chromate of lead. On account of the basic nature of this pigment (high litharge content), it gave marked protection to the metal sheets to which it was applied; in fact, this pigment was given a rating of practically 100% by every inspector at every annual inspection. At the end of 5 years' exposure to the sea air, this paint is still in perfect condition, and will probably give excellent protection to the iron plates for many years to come. Panels 9 were painted with that type of neutral red lead which Mr. Sabin claims to be a good protective paint. This neutral red lead paint at the end of 3 years' test gave very poor results on every plate to which it was applied. The surface whitened, chalked, and became covered with little rust spots. The neutral and inert nature of this red lead made it of no more value as a metal paint than ordinary clay. On plates No. 10 there was applied a red lead containing a considerable percentage of litharge. At the official inspection of the tests, this basic litharge red lead received a very high rating, and completely demonstrated its superiority over the neutral red leads. Mr. Gardner.

If the engineer is to adopt red lead as a primer for structural steel, he should specify that the red lead used, if in dry form, should be mixed on the job in oil, or if in prepared mixed form, should be a red lead of inhibitive properties. In the speaker's opinion, it should contain not less than 80 or more than 90% of Pb_3O_4, the remainder being litharge. Most high-grade red lead will run between 82 and 90% of Pb_3O_4. The red lead used should be extremely fine; 99% should pass through a 200-mesh screen. It should contain not more than 0.5% of impurities of any type. Engineers who are responsible for the upkeep of steel bridges, railroad equipment, and similar structures on which red lead is used, are rapidly becoming convinced of the value of basic inhibitive red lead. Specifications issued by the Union Pacific and Southern Pacific Railroads, and the City of Pittsburgh, call for a pure red lead having a minimum of 85% of Pb_3O_4. Specifications of the Rock Island System, the Illinois Central Railroad, and the Yazoo and Mississippi Valley Railroad call for a minimum of 80% of Pb_3O_4. The Public Service Commission of the State of New York has also issued specifications for a red lead having a minimum of 85% of true red lead. This red lead will be mixed with iron oxide, silica, asbestine, and similar pigments, and made into prepared paints. These paints will be used on the new elevated and subway roads.

That thoroughly inhibitive pigments are necessary for the protection of steel cars is evident from the practical tests made at the Pullman shops in Chicago, where thousands of steel cars are produced annually. The following quotation from a paper on this subject, as prepared by the chief chemists of the Pullman Company, presented at a recent meeting of the Master Car and Locomotive Painters' As-

Mr. Gardner. sociation is quite illuminating, in view of Mr. Sabin's statements regarding rust inhibitive paints:

"The pigment must be of such material that it will be inhibitive, or, in technical words, must be alkaline to a slight extent and positive electrolytically to the steel or preferably a combination of both those factors. The Pullman Company, through their laboratory, devised a very comprehensive and careful series of tests, taking into account all the above factors and many more of slightly less importance. All these tests have checked up remarkably well with outdoor exposure tests on steel panels."

As this reference indicates, rust inhibitive pigments of the basic type are being used widely for the protection of metal surfaces. For this purpose, there are many pigments which have proved quite as suitable as the best grade of red lead. Among these may be mentioned basic sulphate of lead (blue lead), zinc oxide, and the red and black forms of iron oxide. Prepared metal paints made from such pigments have proved highly efficient in test and in practice.

In summing up this discussion, the speaker wishes to impress on the engineer the necessity of using, as a priming paint for metal, one which will contain thoroughly inhibitive pigments, either of the basic or chromate type. Manufacturers of lead products may find that they can economically work up their off-color, low-grade, white lead for producing neutral red lead of 98% tetroxide content. Such a product, may possibly be used in the manufacture of rubber goods, glass, or pottery, but is entirely unsuited for application to steel. To recommend such an inert pigment for the protection of iron, shows either a total ignorance of the well-recognized principles of metal protection, or an absolute disregard for the principles of conservation. In the speaker's opinion, it is quite as criminal to recommend a worthless paint for use on the steelwork of a modern structure, as it would be to allow the use of structural steel which is physically imperfect. In either case the ultimate result would probably be the same.

Mr. Carpenter. A. W. Carpenter, M. Am. Soc. C. E.—In his brief and well-written paper, Mr. Sabin has given a great deal of very interesting information, from his point of view, on some of the newer theories and practices regarding oil paints for the protection of ferrous metals. He has used the opportunity to deprecate certain theories he does not himself accept, and devotes a large part of the paper to red lead, the paint pigment in which he is especially interested. He states that "when a new theory is offered, some of them [referring to paint chemists] make a great rejoicing over it without first finding out whether or not it agrees with the facts."

In spite of the speaker's sincere respect for Mr. Sabin's knowledge and experience, he is tempted to suggest that he is indulging in a little rejoicing himself over this new red lead theory, without offering much

Mr. Carpe

proof as to agreement with the facts. He states (1) that it is now known that the value of red lead depends on the quantity of true red lead it contains; (2) that the finer the red lead, the better it is; and (3) he suggests that red lead with more than 6% of litharge, when mixed with oil, will set in a day or so. This is evidently offered in opposition to another new theory which has been advanced, which is, that a certain proportion of litharge, at least in the form existing in the red lead as a result of the process of manufacture, is a benefit to the material as a paint pigment. The only evidence Mr. Sabin offers in support of his claims is the reference to the tests on the Havre de Grace Bridge, and this is not very satisfactory, as he does not state the litharge content of the two red leads used straight, and there might be some question as to the influence of the pulverized silicate on the third red lead mentioned. On the other hand, the only evidence offered in support of the theory that litharge is a valuable element in red lead, as far as the speaker knows, is the Atlantic City tests, in which ferrous plates coated with paint made with a red lead containing a proportion (stated to be about 12%) of litharge, were better protected after 5 years' exposure than similar plates coated with paint made with orange mineral, which contained practically no litharge.

The speaker's principal object in discussing this paper is to suggest that perhaps both these contradictory theories are wrong, or, at least, not worth the fuss that is being made over them. This suggestion is the result of a series of tests of samples of red lead of varying composition, as shown in Table 1.

TABLE 1.—TESTS OF RED LEAD FOR NEW YORK CENTRAL LINES BRIDGE ENGINEERS' COMMITTEE.

No. of sample.	Chemical Analysis.			Fineness.		Spreading Capacity of Paint.					
						24 lb. lead to 1 gal. oil.			30 lb. lead to 1 gal. oil.		
	True red lead. Percentage.	Litharge. Percentage.	Other elements, approximate percentage.	Relative fineness.	Weight per cubic inch, in grammes.	Square feet covered.	Relative percentage.	Working qualities.	Square feet covered.	Relative percentage.	Working qualities.
1	94.2	5.7	0.2	2	27	23	74	O. K.	30	79	O. K.
2	92.9	7.0	0.01	3	33.6	20	65	Covered poorly and brushed out hard.	29.5	77	O. K.
3	87.7	11.7	0.5	1	23	28	90	O. K.	33.5	88	O. K.
4	84.5	15.2	0.3	3	33.6	31	100	Mixed thin and sagged.	38	100	O. K.
5	80.6	19.1	0.3	1	23	20	65	Brushed out well but ran.	27	71	O. K.

Quantities for spreading tests: ¼ pint of oil to 12 oz. and 15 oz. of lead, respectively. Spreading tests made May 18th and 22d, 1911.

ater.

This table shows the results of the examination and tests of five samples of red lead, each furnished by a different manufacturer. It shows the fineness of division, the chemical composition, and the spreading capacity when mixed with linseed oil in the usual paint proportions. It will be noted that the true red lead content ranges from 80.6 to 94.2% and that of litharge from 5.7 to 19.1 per cent. The fineness determination was made by the weight method, and is merely a comparative one based on the theory that the fineness of division of materials of equal specific gravity and similar structure of particles will vary inversely as the weight of equal volumes of the materials when equally packed. The weights given were determined by sifting the leads through a 20-mesh wire screen, placed at a fixed height above the measure of volume until the latter was filled, striking the material off flush with the top of the measure, and then weighing the contents. The lead with the lowest true red lead content (Sample 5) was found to be a tie with one other as the finest in division of all five leads; and the lead having next to the lowest true red lead content (Sample 4) was a tie with one other, the one with next to the highest true red lead content (Sample 2), as the coarsest. The other cases exhibit a similar lack of law of relation between true red lead content and fineness of division.

The paint mixtures for the spreading tests were made with open-kettle boiled linseed oil and no drier. They were applied to clean, galvanized, sheet-metal surfaces—well weathered and apparently all in the same condition—by one operator, on the same day, and under the same weather conditions, using a separate clean brush of one size and kind for each paint. The quantity of paint was small, but covered an appreciable area in each case, and the results with the two proportions of lead and oil will be noted as generally consistent. They show that one of the two coarsest leads (Sample 4), and the one having next to the lowest true red lead content, gave the greatest spreading; and one of the finest leads, which had the lowest true red lead content (Sample 5), gave the smallest spreading.

It seems safe to say that, if these tests are a correct indication, there is no definite relation between the fineness, the proportion of true red lead, and the spreading capacity of the pigment in oil, but that other elements control.

The foregoing determinations, of course, do not give any indication as to the durability of the respective leads in paint, nor their respective values for the protection of ferrous metals from corrosion. In the endeavor to determine these qualities, the leads were mixed with boiled linseed oil in the proportions of 20, 25, and 30 lb. of lead to 1 gal. of oil, and the resulting paint was applied, under three different conditions, in one coat each on steel plates 5 by 7 in. in

lateral dimensions. The paint was prepared by first pouring a small proportion of the oil over the full quantity of red lead and letting it stand over night. Next morning, the remainder of the required quantity of oil was added, and the whole was stirred thoroughly. M Carpe

Three lots of paint, one for each proportion of 20, 25, and 30 lb. per gal. of oil, were thus mixed from each of the five samples of red lead, thus making fifteen lots. From each of these fifteen lots a steel plate was coated and properly marked for identification. Then from each lot, two small wide-mouthed bottles were partly filled. One of each of these bottles was allowed to stand open, without stirring, for a period of 2 weeks, at the end of which time the paint was stirred and applied to a steel plate in a single coat; in no case had the mixture set so that it could not be readily stirred and applied in a good coat, although in one or two cases the coats were quite thick. The paint in the second set of bottles was stirred every day for 2 weeks and was then applied to steel plates, except that one lot had set so that it could not be applied; that one was made from Sample 5 in the proportion of 30 lb. of lead to 1 gal. of oil; this sample contained the greatest proportion of litharge, 19.1 per cent.

After coating these plates, they were exposed to the atmosphere on the roof of an office building in New York City and left for 463 days, or 1 year and some 3 months. They were then brought indoors and examined with the following result:

The plates to which paint was applied immediately after preparation were in generally good condition. Those coated with 25 lb. to 1 gal., were in the best condition, although the coatings had changed color. There was no choice between the coatings with leads made by the different manufacturers. The 30-lb. per gallon coatings were too thick in some cases, and showed more breaks, with rust spots, than the 25-lb. per gallon coatings. The 20-lb. per gallon coatings showed the best color, probably due to the greater protection of the pigment by the oil.

The coatings with the mixtures which stood 2 weeks before application, in many cases were as good and had protected the steel as well as those applied immediately after preparation; in some cases it was evident that the mixtures had commenced to thicken or "set" at the time of application.

The coatings applied after the mixtures had been stirred daily for 2 weeks gave good protection, with two or three exceptions. They generally were not quite as good as those from the paints which stood 2 weeks without stirring, and the latter were not quite so good as those from the paint applied immediately after preparation. In several cases the paint had begun to thicken up or "set" when applied, but the results from these coatings were decidedly good.

ter. In view of the foregoing there does not seem to be anything to choose between the different leads, as affecting the durability and protective qualities of paint coatings in which they are incorporated. For economical reasons, the choice would seem to lie with the one that spread the farthest, but there does not seem to be any way to predetermine the spreading capacity of paint made with any particular lot of red lead, from the fineness of division or litharge content of the lead, at least within the range exhibited by the samples tested.

It may be thought that the exposure tests were not long enough for comparative results. In this regard attention is called to the fact that the tests were made on single coats, and that the time was rather long for such.

It is not the intention to offer these tests as in any way conclusive, but simply as some definite determinations, as opposed to mere statements of opinion. It is hoped that others may take up the investigation, to the end that the facts may be established. It would seem to be a good field for investigation by the National Bureau of Standards.

Mr. Sabin's paper would lead one to think that red lead is the only good pigment for paint for preserving ferrous metals. Although it is undoubtedly one of the best of the practical pigments for use under many conditions, there are other conditions under which it is not satisfactory; and it is doubtful if its use in more than one coat is desirable on structural steel for ordinary exposure, owing to its cost, its susceptibility to deterioration in color and integrity in contact with gases, and its often objectionable color.

It may be very well argued that one or two under coatings of red lead paint will be generally improved by over coatings of paints with other and cheaper pigments, rather than additional coats of red lead. The speaker has also known of many instances in which oxides have made remarkable records for durability and protection in paint coatings applied directly to steel surfaces, and, he has a much higher regard for the value of certain iron oxides for use in structural steel paints than the author expresses for this class in general.

Mr. och. MAXIMILIAN TOCH, ASSOC. AM. SOC. C. E.—This paper can hardly be said to throw any new light on an old subject; an analysis of all that it contains can be summed up as a propaganda for an alleged new material known as red lead ground in oil, and sold in paste form.

Mr. Sabin's conclusions with reference to the condition of the paints on the Havre de Grace Bridge and the final report are incomplete. The report really describes as excellent nine paints, three of which are red lead; and one of these is a ready-mixed red lead containing, as he says, 15% of a pulverized silicate. Now, to be frank, such an indefinite term as "pulverized silicate" should not be used.

The paint in question contained a substance known as aluminum silicate, but better known to engineers as ordinary American clay. The statement that 15% of the composition of this paint was clay is also misleading. Mr. Sabin, however, has corrected this by stating that the percentage of clay is nearer 25. The specific gravity of clay is a little greater than 2, the specific gravity of red lead is about 9. The volume of these two materials is such that the actual quantity of clay in this paint was nearly as much as that of red lead. In other words, 25 lb. of a good, clean clay has the same bulk as 85 lb. of red lead. Mr. Toch.

Now, what has the author proved? That the two pure red leads in these three results were only as good as the red lead which was mixed with clay, and, therefore, that clay, when mixed with red lead in moderate proportions, is just as good as red lead, for, after 6 years, there has been no difference in these three test plates. Then again, every one of these red leads was coated over with a carbon paint, and, if the test proves anything, it proves that the carbon paint was a perfect excluder of oxygen and water, and therefore no rust resulted.

The speaker has examined the report of the Havre de Grace Bridge committee very thoroughly, has been active on this committee to some extent, and has inspected these plates. Mr. Gardner has stated that among the paints which have turned out the best was that on Plate No. 12, and as nearly as can be determined from the report, that plate is the only one having two coats of paint, all the others having three coats; and the 600-sq. ft. spreading test is as good as the best. The speaker is not ashamed of this, in view of the fact that it was a paint invented by himself and made by his firm. Bearing out Mr. Gardner's statement with reference to the inhibitive quality of the basic paints, that on Plate No. 12, according to the published analyses, was the only one which was alkaline, due to the fact that the priming coat was a now fairly well known cement paint. The finishing coat was a carbon paint, and after 6 years these two coats are equal to the three coats of the others.

Four smokestacks for the power-house of Long Island City were painted in August, 1906, and are in as good condition to-day as they were 7 years ago. In that case three coats were applied, but the first was painted directly on rust, and that rust has never progressed sufficiently to scale the succeeding coats of paint.

The author has not even "damned by faint praise" any other material as a paint. On December 13th, 1913, the speaker examined very critically the test plates at Atlantic City which were painted in 1908 with single pigments, where, for instance, oxide of iron, red lead, graphite, etc., were applied without any subsequent protective coat, and he must take issue with Mr. Sabin, for the three red lead plates which are still on exhibition there are not as good as the three plates

Mr. Toch. covered with Prince's metallic brown, which is an oxide of iron; nor is the American orange mineral, which is a pure form of red lead, anywhere near as good as the sublimed blue lead. In fact, to be perfectly frank, the sublimed blue lead is far better than either red lead or orange mineral.

Allusion must be made to Mr. Sabin's statement: "one should not be disturbed by talk about 'newer paint oils'." The author did not mean "newer paint oils" when he put those words in quotation marks, but intended to refer to "newer paint materials," which was the subject of a lecture delivered by the speaker in London nearly 3 years ago, and the speaker is quite sure that Mr. Sabin is not so much against progress as this statement would indicate. Considerably more than 100 000 copies of that lecture were reprinted, and it has been quoted extensively both in Germany and in France, therefore, when Mr. Sabin pretends that there are no newer paint materials, the speaker is not inclined to take him seriously.

The speaker has always held the strong opinion that no man should take out of the common store any more than his allotted share. To him it is just as criminal to corner the wheat market and raise the price of bread as to take money out of the pockets of other people; there is this difference, however, the pickpocket can be prosecuted, but the man who raises the price of bread can not.

About 4 years ago, at a time when the crop of turpentine was normal, some men in the South cornered the market. Hundreds of thousands of gallons were stored, and none of it was sold until the price, which should have been about 45 cents, was raised to $1.13 per gal. The speaker very quietly cast about for a substitute for turpentine, and so did many others. There was one man, in the employ of the United States Navy, who showed much more courage than any admiral ever did in battle, when he wrote a specification for turpentine substitute for the United States Navy, and in one year reduced the purchases of turpentine in the Navy about 70 000 gal. A large number of other progressives did the same thing, with the result that to-day we can do without turpentine absolutely, and the price in its tumble shattered those who had raised it abnormally.

About 3 or 4 years ago, through the failure of the flaxseed crop in the United States and other countries, the price of linseed oil rose from 36 cents to $1 per gal. Although the speaker has never contended that he had found an oil which was just as good as linseed, he has proved that there are several which, in conjunction with linseed oil, make most excellent paint. If the author thinks there is nothing in the newer paint materials, let him ask the manufacturers of printing ink, linoleum, artificial leather, and also a number of paint manufacturers, whether Soya bean oil and Menhaden oil are not valuable assets to them to-day. A few months ago one of the largest manufacturers

of Menhaden oil stated that where he sold one barrel to the manufacturers of paint and linoleum 10 years ago, he is selling a thousand barrels to-day. In the speaker's lecture on fish oil he pointed out that many of the failures were caused by the ignorance of the men who made the tests. On any price list of fish oils to-day will be found whale oil, porpoise oil, and seal oil, under the heading of fish oils, and it was the speaker who pointed out to the paint industry that these are animal oils and not fish oils, and, further, that a fish oil, to be suitable for paint material, had to possess certain constants, otherwise it would be useless. Mr. Toch.

In 1896 5 gal. of China wood oil came to the United States. As far as known, it was the only China wood oil imported that year, and the speaker received those 5 gal. During the year ending October 31st, 1913, 5 400 000 gal. of China wood oil were consumed in the United States. This oil, before it was understood, had a very bad reputation as an adulterant and an undesirable paint material. When one talks about adulterants one is treading on very thin ice. Everybody is familiar with the high-priced Epicurean dish known as terrapin. To serve terrapin at $3 per microscopic portion is considered in high society a great feat, and yet, 150 years ago, when a good slave was sold in Maryland, the agreement generally made was that the slave must not be fed on terrapin, for at that time it was considered hardly good enough food for dogs. If China clay, which is the pulverized silicate mentioned by the author, were equal in price to red lead, a feature would be made of its use instead of a secret.

This paper tends to show that there is a new material on the market known as red lead ground in paste form, which can be very easily thinned and made ready for use. Pure red lead, ground in pure linseed oil, in paste form, has been known for many years, and, what is more, the author knows it has been known, because orange mineral, which is a pure form of red lead, has been used by color grinders as far back as the speaker can remember. The speaker made ready-mixed red lead more than 20 years ago, and made it successfully; and it remained in its ready-to-use form. He has no quarrel with red lead, because he thinks it is a very good material. It is not better than sublimed blue lead, nor is it better than certain forms of oxide of iron. It is certainly very much dearer than any other paint, and, on account of its great weight, it is harder to apply than a lighter one; it does not stop progressive oxidation, nor is it a fit material for use on steel which is to be bedded in concrete, not because of the red lead, but because of the oil, which is readily saponified.

Everybody must admit that in some places red lead is a most excellent protective paint, but in other places it does not give such a good account of itself. Then again, every paint chemist knows that there are some red leads that contain nearly 1% of caustic soda, which

Mr. och. is quite sufficient to destroy a paint when it comes in contact with water, and all know that the red lead which can be ground in paste form and remain plastic is not the red lead that has been used heretofore, which contains litharge, and the old litharge red lead paint should not now be condemned because it forms a cementitious material.

As a paint under water, red lead is sometimes good and sometimes bad, depending, of course, on whether or not it has been made by the nitrite of soda process.

A large quantity of red lead was used on the steel of the locks of the Panama Canal as a priming paint, three coats being applied in many places. When submerged in the lake water, the red lead turned blue, owing to the action of the sulphur gases on the lead, and the paint was in many places entirely destroyed. Whether this was due to the fact that it contained a small percentage of free caustic soda and thereby became soluble in water, or whether the decaying organic matter destroyed it, amounts to the same thing. Preliminary tests, however, may have demonstrated these defects.

The speaker has at all times stood for fairness and the exact representation of facts with regard to paint materials. He has spoken well of all those materials which his judgment and experience have taught him were good. He has even had the temerity to talk well of his own paints whenever he has had the chance, for the world is large enough to support all, and to give every one a share. He is glad of the opportunity to state that he and Mr. Sabin have been friends for many years, and he respects him highly. In differing in opinion with him, he is simply breaking a lance in the interest of paint science. The author is a man who is well equipped, and has had a long and varied experience which entitles him to the right to talk, but the speaker agrees with the poet Goethe, *"Der Mensch verhoent was er nicht versteht"*, which, translated into good "American", means "A man is down on a thing when he is not up on it".

ir. mbs. R. D. Coombs, M. Am. Soc. C. E.—The manner in which paint is ordinarily applied in commercial practice is so irregular and faulty as to render any direct comparison inaccurate. The painting, as usually done on structural steelwork, is about the poorest job on the entire construction. As most erecting engineers know, the work of painting is disliked by the higher priced men—the iron-workers. Instead of having the work done by a $4 or $5 iron-worker, it is done by a $2 to $3 steel painter or apprentice, and it is frequently a very poor sample of painting. Again, in the entire painting, the shop coat would seem to be the thing which can most easily produce a poor result.

The average paint used in the shop is a relatively cheap material, costing about one-half as much as a good field coat, and it is frequently put on by the cheapest labor in the shop, and more frequently it is put on very badly. Now, it would appear reasonable to expect that

a poor shop coat over an imperfectly cleaned surface would have a very great bearing on the final net result of the work. Thus, if a $0.60 paint is put on in the shop as badly as is frequently the case, and then a $1.25 paint is used in the field, it seems open to question how much of the result of the final painting is chargeable to the shop coat and how much to the field coat. Mr. Coombs.

Referring to the matter of red lead paint as practically applied—not as a laboratory test, but as done in the field—the question arises, how many pounds of red lead to a gallon of oil should one expect low-grade painters to use successfully. There are in existence a number of specifications, issued by individuals and corporations of high standing, calling for 33 lb. of lead to 1 gal. of oil. On a rather important piece of construction, in the vicinity of New York City, the master painter, who was in the employ of the company and had no object whatever in cheapening the work, reported that he could not put on as much, unless he was allowed to use a trowel. This may have been an exaggeration, but it is the speaker's recollection that about 22 lb. was as much as the men found they could work with to advantage.

The speaker does not wish to convey the impression that the painting in the field is a much better piece of work that that done in the shop, but that the painting, either in the shop or in the field, as usually done, is not a good job. Further, the manufacturer alone is not to blame, but also the owner and the engineer. It is simply a bad practice which has become very common, and for which all concerned are at fault. Reference is not made particularly to red lead paint, but to all paints.

There seems to be some tendency on the part of the manufacturer to regard the multitude of paints now specified as a senseless nuisance, and—it must be admitted—with perhaps some justification, from his standpoint. However, having recovered from his chagrin at the receipt of an order embodying another change in paint from the kinds in immediate use in the shop, and after faithfully incorporating the proper instructions in the shop orders, he often takes no further interest in the matter. The paint question is now up to the shop foreman, whose theories thereon are often in need of expurgation as well as elaboration. To the minds of many shopmen, who take a real pride in doing good shop work, any old paint of the specified color is good enough.

Turning now to the paint man, it is difficult to understand how he can escape responsibility for at least part of the skepticism of the manufacturer, engineer, and owner, as many paint salesmen rather permit the impression to get abroad that the paint used by their competitors will eat the steel rather than protect it.

The private owner is not so much to blame, as he is neither an engineer nor a paint expert. Those corporate owners, however, having engineering departments, whose duty it is to conserve their construction interests, can hardly be absolved from blame for careless work.

Mr. Coombs. The speaker is unwilling, although able, to specify definite instances, in which a cheaper grade of paint than that specified has been used, or in which either, or both, the shop and the field painting have been very badly done. Painting in wet or freezing weather, the use of liberal quantities of "thinners" and benzine, painting over scale, rust, cinders, mud, etc., and the omission of paint, are not such rare occurrences as optimists would have us believe.

It would seem that it is about time for a reformation in the methods used in painting. Not so much in relation to the work done under efficient inspection as in the great bulk of work included in small contracts throughout the country, which are either poorly inspected or not at all. Where paint is really needed, it is usually badly needed; and, unfortunately, the work on which the future maintenance will be inadequate is just the work on which the original inspection and superintendence is inadequate.

It is the speaker's opinion that, in a great many cases, particularly for interior steel beams encased in concrete, no field paint whatever is necessary, and that the expense of painting such members would more properly be expended in the better painting of the members which really need it. Further—although it may seem contradictory to the previous remarks—better results might often be obtained by using a $5 man and $1 paint, than a $1 man and $5 paint.

As a result of some years of experience with a class of construction in which there are very thin members subject to an extreme and variable exposure, the importance of a revision in the common attitude toward the paint question appears to be of considerable importance. Such work may be regarded as an accelerated test, and as developing in a few years what may be expected in a longer period in heavier material, under incompetent supervision.

The speaker would like to ask Mr. Rights what he thinks the average price of shop paint is throughout the country, that is, considering the average of the total number of shops, not the average of the total tonnage.

Mr. Rights. LEWIS D. RIGHTS, M. AM. SOC. C. E.—The speaker, as a member of the American Society for Testing Materials, has attended the conventions at Atlantic City, has inspected the paint fence, and has listened to discussions regarding the values of inhibitors and stimulators. He has followed Mr. Sabin's paper and the discussions with a great deal of interest, with the hope that something would be brought out to help to clear up the situation, from the standpoint of one of the parties who is as much interested as any of the rest, if not more so. He refers to the bridge and structural manufacturer, who has the responsibility for the application of the paint, both in the shop and in the field, and is therefore a very important link between the paint manufacturer and the engineer and owner.

It may not be out of place to mention the conditions under which painting is done at the shops of the structural and bridge manufacturers. So far as the speaker knows, very few of the bridge shops have any adequate facilities for doing their painting inside a building, where the work can be done regardless of wet or freezing weather. Although it is true that during the most severe winter conditions painting is done occasionally in the finishing end of the shop, facilities there are so cramped that the quantity of painting done under cover may be said to be almost negligible. It is safe to say that practically all the structural steel painting is done in the open. Mr. Rights.

Some years ago, the owners of one of the largest shops carefully considered plans for covering their shipping yards and protecting them so that painting could be done at all times. The item of expense, however, was such that it was decided that the condition of the business would not warrant increasing the cost of painting to the owners.

The bureaus which inspect a large proportion of the fabricated steelwork have a rule that the finished material must not be painted until their men have had an opportunity to go over it. Frequently, it takes an inspector several days to cover the shops in his vicinity, and this means that the steel must sometimes lie in the open for a day or so before it can be painted. Now, considering that painting cannot be done in either wet or freezing weather, nor can it be done until the work is inspected, it will easily be seen that the time allowed for doing the painting is very much reduced, and therefore, of necessity, the application of the paint at the shop must be a somewhat hurried job. In general, the class of labor doing the shop painting is fully as good as that employed in the other details of manufacture, and considerable care is exercised in endeavoring to see that all parts of the pieces are evenly covered.

In connection with the erection of structural steel, it has become the practice of many of the bridge manufacturers to sublet the painting to men who make a specialty of work of this class. It has been found that bridge erectors do not relish handling a paint brush, and better results have been secured with specially trained men. As the manufacturers are interested in producing a high grade of work, it has become the custom for them to buy the paint and ship it to the job in unbroken packages. It is then sold to the painting sub-contractor at an agreed price per gallon, and the work of applying the paint is inspected either by the owners' inspectors or by some one appointed by the bridge manufacturer.

The speaker would like to state here that the bridge manufacturer is not in favor of cheap paint. The fact of the matter is that he would rather put on a good paint, provided the cost of this good paint has been properly covered by the specification, and figured in his original bid. It will easily be seen that the more work the bridge manu-

Mr. ights. facturer can put into a job, the greater will be his percentage of profit, and that, in addition, he will produce a more sightly structure. The bridge manufacturer carries at his shop several lines of standard paints, which are probably as good as can be bought under ordinary specifications, such as red lead, white lead, iron oxide, and some form of carbon or graphite. He is entirely willing to apply the best of these, as called for, or to provide a better paint, if required. What he most desires, however, is that the discussions which have been going on among engineers for the last few years will produce something in the way of standard kinds of paint suitable for each class of work. As matters now stand, it would seem that almost every engineer was working out his own theories of inhibition and stimulation, and that each one had a different variety of paint. Of necessity, the bridge manufacturer must order a somewhat larger quantity of paint than is actually required for each particular structure, with the result that the paint shop has the appearance of a medicine shelf, containing left-overs, each one strongly advocated by one individual engineer, and strongly condemned by all the rest of them. These left-overs will not remain fit for use very long, and are frequently a total loss to the manufacturer.

Years ago, a similar condition applied to specifications regarding the quality of structural steel. To-day, the manufacturer and the engineer have been able to get together, so that the same grade of structural steel is used for railway and highway bridges, buildings, and cars, and when the total tonnage is considered, the special-grade steel is a small factor. Cannot the paint manufacturers and the engineers get together on some such broad basis, and settle on paints which will be serviceable to the owner, without undue cost, which can be readily applied by the manufacturer, and which will fit 90% of the steel structures. " 'Tis a consummation devoutly to be wished".

Mr. lcher. W. E. Belcher, M. Am. Soc. C. E. (by letter).—This subject is certain to hold the attention of a large proportion of the members of the Society, for all are looking for more information. The writer, however, confesses to a feeling of disappointment in reading the paper, particularly in regard to the very small number of paint materials selected by the author for consideration, and also because of his failure to cite from his own wide experience definite records of paint performances from which engineers might be assisted in forming their own conclusions.

The writer would like to add the following random notes from his experience:

For several years a red lead paint was always specified by him for structural steelwork for buildings, following the example of many

Mr. Belcher.

railroad and Government specifications. When inspecting the work done under practical shop conditions, the paint was found to be more or less unsatisfactory. There was a decided tendency for the formation of a hard sediment at the bottom of the paint bucket, and ordinary paint laborers never stirred the paint with sufficient frequency to obtain a uniform coat; this would upset fine calculation as to the number of pounds of lead per gallon of oil. There are on hand in most bridge shops several kinds of red paint which can hardly be distinguished from one another immediately after application, and might easily be substituted for one another—unintentionally perhaps—without subsequent detection. The writer has seen on detailed shop drawings the paint note, "One Coat Red," though the original specification had been quite elaborate. If specifications could be reduced to this simple form there would be no difficulty in getting what is called for. Another point which seems detrimental to the use of red lead is the fact that its color soon becomes dull after exposure, without regard to the number of coats applied.

The writer has always been prejudiced against graphite paints, on account of their extreme thinness and also because of the very conflicting arguments advanced by the manufacturers of the pure pulverized graphite paint, on one hand, and those of the mixed silica and graphite, on the other. A specification calling for Prince's mineral and for iron oxide to which graphite was added in small quantity for smoothness and color, was tried out with good results, and has since been followed. It overcomes very largely the objections above offered to red lead, and has generally good lasting qualities.

The preparation of a ground red lead paste mixed in oil would not appear to be entirely new, as one of the large paint manufacturers in Chicago submitted to the writer about three years ago a preparation called "Liquid Red Lead." This was a concentrated red lead paste to which, if memory serves correctly, a small quantity of silica was added, which assisted in an effective way against sedimentation.

This material was submitted in connection with a study of the subject of gas-holder paint. One manufacturer, submitting samples at that time, offered "Special Gas-Holder Red" which was recommended to fill every possible requirement, although the description of its manufacture was very vague. Investigation disclosed the fact that the manufacturers were as little certain of the desired ingredients of a gas-holder paint as the writer was, and, consequently, they put in a little of everything, including old house-paints, surplus paint stock, kettle scrapings, paint skins, etc., with enough red to color the mixture.

The writer is a firm believer in the theories of Messrs. Cushman and Gardner in regard to the corrosion of iron and steel; at the same time he must agree fully with Mr. Sabin that it is practically impos-

Mr. cher. sible for an engineer to use them in their present state as a guide for intelligent selection of a paint material. Has any manufacturer or manufacturer's representative ever been forced to admit that his particular combination of oils and pigments has been in any way discredited by these theories and experiments? The author's remarks regarding lampblack illustrate this point. We have learned that lampblack is in itself injurious. The manufacturer then states that he has ground it in a certain way which changes its action entirely.

The science of engineering is largely empirical, based on observations and experiments. The present discussion in regard to painting structural steel emphasizes the incompleteness of our record of experiments in this particular branch of the science. In the present situation, our rules are still in the nature of theories and deductions from limited personal experience.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

By Messrs. L. H. Nishkian and William Cain.

L. H. Nishkian, Assoc. M. Am. Soc. C. E. (by letter).—On page 2083† the author computes the shear in section $N\ I$, Fig. 4. An important function of the inclined bar is to relieve the shearing stresses in section $N\ I$, but the author does not take into consideration the vertical component of the stress in the inclined bar. In order to have equilibrium of all the vertical forces acting on that portion of the base to the right section $N\ I$, the vertical component of the stress at section $N\ I$ in the inclined bar should be deducted from the total vertical load, 18 333 lb., and the remainder will be the shear in the concrete in section $N\ I$. Mr. Nishkian.

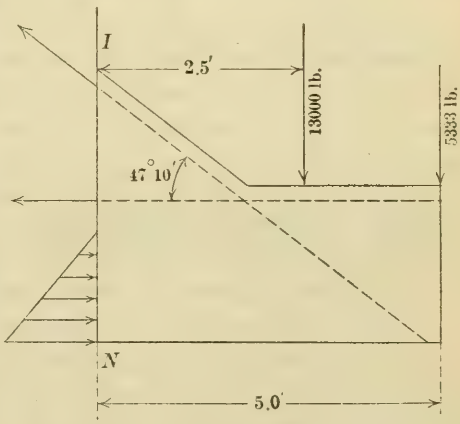

Fig. 12.

The sum of the vertical components, per foot of width of the stresses in the inclined bars, is

$$17\ 750 \times 0.765 \times \frac{3}{2} \times \sin.\ 43° \ 10' = 14\ 000 \text{ lb.}$$

and

$$18\ 333 - 14\ 000 = 4\ 333 \text{ lb.}$$

Therefore the maximum shear in section $N\ I$ is $47.6 \times \dfrac{4\ 333}{18\ 333} = 11.2$ lb. per sq. in. instead of 47.6 lb. per sq. in.

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* Continued from January, 1914, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., for November, 1913.

Mr. Cain. William Cain, M. Am. Soc. C. E. (by letter).—The writer would like to make a correction as to the meaning of v in Formulas (17) and (18), as others besides Mr. Janni* may inquire the reason for the statement that "The unit stress, v, acts parallel to N N'". The statement is incorrect; v, in Formulas (17) and (18), is the unit shear on any plane, lying between O and the steel bars, perpendicular to N I.

Thus, suppose such a plane drawn in Fig. 8 and marked A A' (A lying in N I, A' in N' I'), then since the section, N I, was taken parallel to the direction of the loads, at a point, A, where the bending stress is supposed to be zero, there is shear only on the plane, N I, and consequently a shear of equal intensity, v, on a plane, A A', at right angles to I N, by a well-known theorem. The total shear on the plane, $A\ A' = v\ b\ dx$.

For equilibrium, the algebraic sum of the components perpendicular to I N, of the forces acting on N A A' N must be zero. Therefore

$$(C' - C) \cos.\beta = v\ b\ dx,$$

which is the same as the equation just above (17). Hence, substituting this value in the equation derived just before the last, or in,

$$(C' - C) \cos.\beta\ D\ G = V'\ dx,$$

dividing by dx and then taking the limit, as dx approaches zero and V' approaches V indefinitely, it is found that,

$$v = \frac{V}{b\ \overline{D\ G}}$$

just as given in Formula (17).

Thus Formulas (17) and (18) are unchanged in form, but v is now to be interpreted to mean the unit shear on a plane perpendicular to N I, lying between the neutral axis and the steel.

This is the maximum shear, being greater than the shear on any other plane than A A' passing through A; hence it is of special interest to the designer.

* In his discussion, *Proceedings*, Am. Soc. C. E., January, 1914, p. 239.

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PAPERS AND DISCUSSIONS

A STUDY OF ECONOMIC CONDUIT LOCATION.
Discussion.*

By Messrs. H. Hawgood and M. F. Stein.

H. Hawgood, M. Am. Soc. C. E. (by letter).—The location of conduits for irrigation and power purposes should receive the same study as that for a railroad. Both are transportation problems, and the economic value of distance and rate of gradient are relatively as important in the one enterprise as in the other. Their importance in railroad transportation is fully recognized. Mr. Hawgood

Mr. Hickok's paper is the outcome of studies made by him, under the writer's instruction, in connection with a Southern California project. His treatment of the subject is simple, and his conclusions are logical. The unit costs used in the paper are not fully endorsed by the writer, but, inasmuch as they are introduced for exemplification purposes only, they are not debatable.

In arriving at methods of comparing the economic values of alternative locations, the fact should not be overlooked that, however satisfactory the mathematical treatment, the conclusions must be tempered by personal judgment of the risks to which the local environments subject the alternative locations. Experience has shown that the risk of interruption of service is far less for tunnels than for hillside conduits, and weight is to be given to this and other working experiences in making a final selection of routes.

Mr. Hickok does not credit tunnels with any saving in seepage. Seepage from a water channel is governed, not only by the nature of the channel lining, but by that of the material on which the lining rests. No water would pass through a sieve set in impervious material. The sides and bottom of a rock tunnel may not be, and rarely are, entirely impervious, yet their opposition to percolation is of suffi-

* This discussion (of the paper by C. E. Hickok, Assoc. M. Am. Soc. C. E., published in December, 1913, *Proceedings*, but not presented at any meeting) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

good. cient magnitude to aid materially the lining proper, and in most cases to reduce seepage to a negligible quantity.

In his graphical representation of results, the author uses a diagonal line for each combination of the costs of two structures. He gives seven different structures; taken in pairs, twenty-one combinations are possible. Twenty-one diagonals, with their legends, would be found to be confusing and impracticable of application. It might also, and probably would, be found convenient to add other types of structures, which would call for a considerable increase in the number of diagonals.

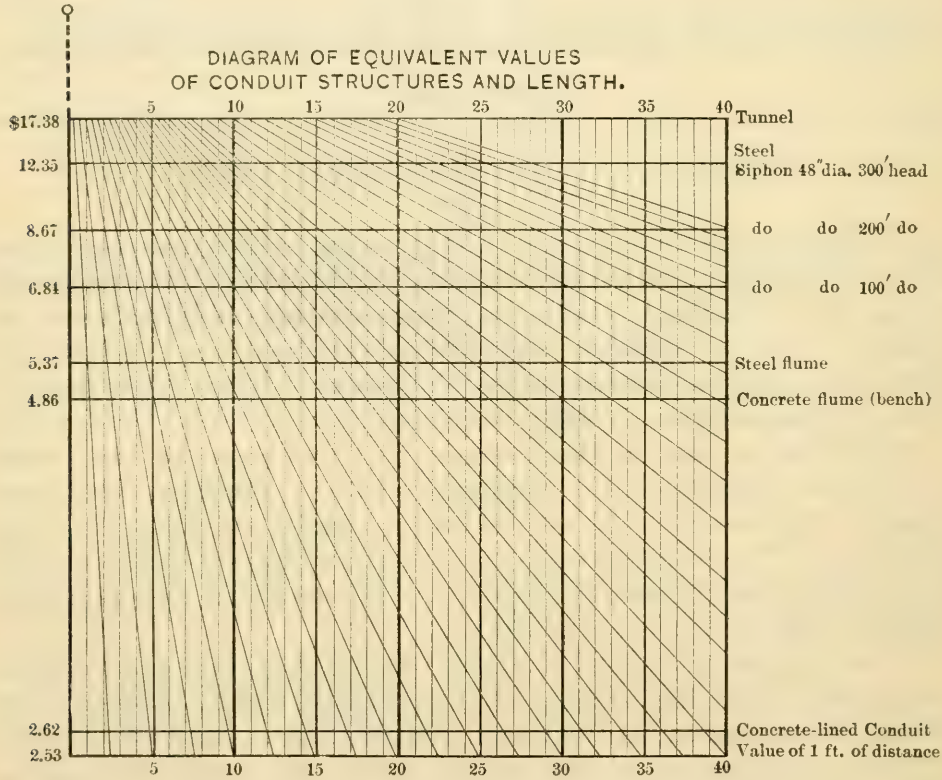

FIG. 3.

The writer suggests Fig. 3 as a more convenient form of diagram and as covering all phases of the subject. The radiating lines permit finding the equivalent distance, or type of structure, in terms of length of any other type of structure. The horizontal lines, each representing a particular type of structure, are spaced so that their distances apart vertically bear the same ratio one to the other as their respective unit costs. To exemplify the use of the diagram, assume that, of two alternative routes, Route No. 1 has 200 ft. more tunnel and Route No. 2 has 1 500 ft. more lined conduit and 300 ft. greater length of line. Find Vertical 3 on the base line representing the economic value of 1 ft. of distance; trace the radial thence up to the concrete-

lined conduit line, and read 289 ft.; add this to the 1 500 ft., giving 1 789 ft., the virtual length of Route No. 2 expressed in terms of concrete-lined conduit. Then find 2 on the tunnel line (top of Fig. 3), trace the radial down to the lined conduit line, and read 1 324 ft., which is the virtual length of Route No. 2 expressed in terms of lined conduit. A comparison of this value with that of 1 789 ft. for Route No. 2, decides in favor of Route No. 1. Mr. Hawgood.

Mr. Stein. M. F. Stein, Assoc. M. Am. Soc. C. E. (by letter).—Referring to Fig. 1, the writer notices that in the "Cement-lined Canal," "Concrete Flume," and "Tunnel" sections, the author has used a thickness of concrete of 3 in. It would seem, from the writer's experience and observation, that though this thickness may be theoretically sufficient to withstand the water pressure, it may not give a sufficient factor of safety for various contingencies which cannot be calculated. It is generally difficult to obtain water-tight construction with a thin wall, using the customary type of labor and superintendence in mixing and placing the concrete, and frost action may prove destructive in a cold climate, especially in the event of seepage through the concrete. A thin section, furthermore, provides little safety against cracking, should there be any unequal settlement. These points would apply with less force in a warm and arid region, but the author does not limit his discussion to such conditions.

As regards the tunnel lining, though this may be sufficient for very firm rock, where it is only desirable to obtain a smooth surface, for hydraulic reasons, it would not offer much resistance to the rock movements, shelling, and thrusts experienced in limestone, shale, or clay formations. This is exemplified in the recent failure of the Montreal conduit, under an apparent pressure much less than might be expected in certain kinds of rock, even though the minimum thickness of concrete was 8 in.

Although these arguments affect the diagram (Fig. 2) to some extent, they have, in the writer's opinion, a more important significance. Papers and discussions in the *Proceedings* are widely copied by engineering periodicals, and any diagrams, especially if dimensioned, are taken at face value by many engineers, outside of the Society, some of whom, by training and experience, are incapable of exercising proper discrimination. To such, the sections of Fig. 1 would make an especial appeal, owing to their apparent economy and simplicity, and might be applied without regard to their suitability to conditions which are not those met in average practice.

Relative to this, the writer recalls the failure of a small dam, about 15 ft. high, the design of which was copied from a textbook illustration of a well-known high arch dam, all dimensions being reproduced *pro rata,* with very illogical and inadequate results, further accentuated by the fact that though the original was built in rock, the copy rested on clay.

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PAPERS AND DISCUSSIONS

CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN.

Discussion.*

By A. W. Buel, M. Am. Soc. C. E.

A. W. Buel, M. Am. Soc. C. E. (by letter).—Mr. Gregory, Mr. Janni, and others have brought out most of the points involved, but there remains one of vital importance as affecting the comparison of the hinged and fixed-arch rib as made by the authors. Although they have used the proper form for their hinged ring, their fixed ring is neither correct nor economical, and, therefore, is not a fair comparison. Being segmental, it is subject to great moments at the springing line, as the ring does not conform to any pressure line for any practical conditions. Mr. Buel.

The only economic justifications for the three-hinged ring are difficult foundation conditions and a rise of less than one-sixth of the span with a possible yielding of the abutments.

The authors exaggerate the difficulty of computing the fixed arch, for, by systematizing and tabulating the work, it is quite possible to make the computations in 2 days or less. Moreover, it does not seem to be good engineering practice, nor creditable to the industry of the Profession, to advocate a design because it may save a few hours or even a few days of work for the designer. This is aggravated when there is room for the suspicion that any such saving in the designer's time and energy is largely due to his unfamiliarity with fixed-arch computations.

The advantages of the ribbed ring have been demonstrated and advocated† by the writer in designs worked out in 1899 and 1901.

* Continued from January, 1914, *Proceedings*.
† "Reinforced Concrete," First Edition, 1904.

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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.

Discussion.*

By Messrs. Charles Rufus Harte, W. Kiersted, and George B. Stone.

Charles Rufus Harte, M. Am. Soc. C. E.—Papers such as Mr. Alvord's, which by stimulating discussion tend to spread a broader knowledge of the important subject of valuation, are particularly valuable at this time when the situation seems not unlike that which led the old New Englander, not in sympathy with the party then in power at Washington, to add to his prayer for Divine guidance for the then President the earnest statement "Thou knowest, Lord, he needs it." At the same time, it would seem not entirely out of place here to raise the question whether, in the treatment of the subject, many of us are not beclouding the facts by analyses and discussions which are too academic. Mr. Harte.

The author's definition of "depreciation" includes four widely diverse items of loss of value:

1.—Organic depreciation—the loss of value because of wear of the substance of the elements which make up the machinery or plant—begins with the life of the plant and continues practically uniform to the end. Plotted in terms of time and loss, it is very close to a straight line.

2.—Functional depreciation—the loss of value because elements of the plant get out of correlation—begins with the life of the plant, and, particularly if not met by proper maintenance, accelerates very rapidly until near the break-down point, when it checks and then con-

* Continued from January, 1914, *Proceedings*.

Mr. Harte. tinues slowly to the end. Proper maintenance brings back conditions, but not quite to what they were at the previous take up, so that, plotted in terms of time and loss, functional depreciation appears as a saw-toothed curve.

3.—Obsolescence—the loss of value because the product of the plant no longer meets market requirements as to character—may attack a property at any time, or may not occur at all. Occurring, it may progress with great rapidity; it may progress slowly; or it may progress, halt, and progress again; so that its curve, in terms of time and loss, may be of almost any form. A striking instance of acute obsolescence is that which, almost over night, changed the cable traction system of the Broadway street-car line in New York City from the latest word in traction to a practice not merely obsolescent but actually obsolete. Thus, also, the rapid development of the alternating-current lighting system with high-potential primaries feeding through transformers, the low-potential distribution circuits, in an almost equally brief time made the Edison three-wire underground system practically of the past.

4.—Inadequacy—the loss of value because the product of the plant, though meeting the requirements as to character, is of insufficient quantity—is, like obsolescence, uncertain as to its occurrence and variable as to progress, though not to the same extent as the former; and, like it, its curve, in terms of time and loss, may be of almost any form.

These four elements are so different in character that it is hardly possible to treat them properly together. If the system under consideration is of such extent that the average of conditions is a fair general average, life tables may be applied with very good results, so far as organic and functional depreciations are concerned. Local conditions, however, often make the system average quite different from that of the tables, with corresponding error unless proper correction factors are used. Obsolescence and inadequacy, at best, can only be guessed at.

The author truly says of "the reproduction method, or cost new less depreciation," "This line of evidence is only one source of information as to value." The fair value of the property used and useful for the public is neither the cost to produce nor yet the cost to reproduce; it is the capitalized earning power. It has been argued that for rate-making the earning power cannot be considered, as it is a direct function of the rates to be regulated, and it is hard to escape the conclusion that—at least where there is competition—rate-making must of necessity be an arbitrary procedure. Economic laws, more powerful even than governmental commissions, will compel the practical equalization of rates, even though different values of competing utilities would otherwise permit them to charge different rates for the same service.

In determining physical value, the Courts almost universally—if not entirely so—have held that depreciation must be considered. It is difficult to find logical grounds for doing otherwise. Use—and for that matter disuse as well—causes losses of quality, quantity, and, if extended, of efficiency. That these losses ought to be made good by the owner, or that there exists a fund dedicated to such making good, in no way changes the facts that the loss has occurred, although such outside conditions may offset the physical loss and result in an unchanged or even enhanced value when the two are considered together. Mr. Harte.

With the proposition to determine past depreciation "on the basis of a sinking fund" there will be much disagreement. Past depreciation is a fact which, except as to its rate of progress, is usually ascertainable without great difficulty, and it should be actually ascertained if the determination is to stand investigation.

A depreciation fund properly secured may be included as one of the physical parts of a property, and, if equal to the amount of depreciation, will, of course, offset it, but in such case the depreciation itself must be determined in order to see if the fund is adequate; it is equally possible, unless adjusted frequently, that it may be greater or less. Nor is the writing off of such an item on the books without dedication of the actual money merely an opinion. Properly written off, it stands as a liability of the property, reducing by its amounts the net earnings of the prosperous plant, or adding to the deficit of the losing venture.

Under the caption "Utilities Commonly Require Growing Plants" the author states that extensions produce a developing series of increments, each with its own depreciation. Although this is literally true, the situation is far less complex than the statement would indicate, because much of an extension merges into and thereafter is treated as a part of the older installation, and depreciation is taken as an average on the whole. Building extensions of comparatively small amount, additions to piping and wiring systems, all such are adaptable to this treatment, leaving only the larger individual items to be dealt with separately.

It might well have been said that the earning power of a partly worn plant is almost invariably greater than that of a new one. Smooth operation, and the maximum efficiency, are had only after the very appreciable physical losses due to what Mr. Wilgus, in his recent paper "Physical Valuation of Railroads,"* aptly called the "educational period."

The author states that "One of the reasons the Courts have for favoring the reproduction method, or cost new less depreciation, lies in the fact that such procedure includes appreciation as well as excludes depreciation." It is probable that the attitude of the Courts

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

Mr. Harte. in this matter is due more to the logic of the procedure than to a desire to include appreciation.

The cost of educating the public to the value of the utility, though not attaching to the property in that particular form, remains as a very considerable asset in the going value, which is due to "a fundamental conviction that a utility is really necessary and useful."

In "The Practical Treatment of Depreciation Accounts" the author apparently considers safe only that fund established with some trust company. Aside from the fact that trust companies have failed, and doubtless will do so in the future, such companies find the necessary monies to meet fund interest, overhead charges, and profit, in other securities, and, if the utility by direct investment can save the middleman's profit and overhead charges, and thus reduce the investment necessary to produce the required result, it would seem but fair thus to permit; the matter is purely the question of fact as to whether or not the "securities" are such in more than name, and whether they are a part of the property, or have passed out of it. The first calls for expert financial knowledge; the second must be evident from the books.

A reasonably safeguarded fund or investment for renewal purposes, in a total valuation, should be considered as much a part of the property as the material which it will presently replace. Such replacements for the most part can be made economically only after considerable wear and consequent loss in physical value. If this physical loss is not excessive, and is properly set up on the books as a liability, it is obviously unjust to deprive the owner of its proportion of earnings merely because of its changed form.

The author's first conclusion, that a trust fund to meet depreciation if "properly computed and accurately kept"—this whether in this connection "kept" has reference to the bookkeeping or the actual holding—should be considered a part of the property "used and useful for the public," would seem obvious.

The second conclusion, that the fund must be "actually in hand," is also obvious, if a reasonable interpretation is put upon "in hand." To say, however, that proper book entries do not constitute such condition is, to put it mildly, unfair; carried to its logical conclusion, such reasoning would preclude the consideration, as assets, of any unpaid accounts.

The third conclusion, that "past depreciation may be computed accurately on the sinking-fund basis," is only warranted when "all the facts of the past are known," in which instance there would seem to be little occasion to compute what is already had. It should not be forgotten that, in the Minnesota Rate Case, there were thrown out of Court, because obtained by hypothetical assumptions, figures for land values which were undoubtedly very close to the facts. For the future,

knowledge of the past will often establish correction factors to apply to and make applicable life tables, so that organic and functional depreciation can be predicted with reasonable closeness, but the possible effects of obsolescence and inadequacy can only be guessed at, in the light, respectively, of the development of the art and the local conditions. Mr. Harte.

Conclusion four, that depreciation accounts should be kept by groups having similar life characteristics, and should be revised from time to time, is good, and might well call for considerable detail in the groups. It should be always remembered, however, that such accounts are only estimates, the relations of which to the facts can only be known by actual comparison.

Conclusion five, as stated, that the use of a reserve or sinking fund for private gain precludes its consideration as a factor of value to the utility, is sound only as it applies to such portion of the return from the fund as becomes "private gain." The use of funds set aside for the specific purpose of a reserve or sinking fund in the purchase of securities or in other forms of loan does not destroy the asset unless the security therefor, whether bonds, stocks, or personal notes, loses value; nor if such security pays a rate of interest higher than necessary for the calculated fund, and this excess be withdrawn and otherwise used, does it impair the protection of the remaining portion.

W. KIERSTED, M. AM. SOC. C. E. (by letter).—Mr. Alvord's interesting paper on depreciation appears to be largely a discussion of the paper by Mr. Grunsky,* or at least to have been suggested by the views set forth in that paper. Mr. Kierste

In this discussion Mr. Alvord raises four distinct questions, namely:

> Shall allowance be made for depreciation of physical structures in computing value of a public utility property by the reproduction method for rate-making?
>
> Should a sinking fund, created to take care of depreciation, be allowed to earn a return—in fact, to earn the same rate of return as other portions of the property?
>
> Should a sinking fund thus created be considered as a part of the property?
>
> Should the units of property be grouped according to their respective life limit in accounting?

A difficulty at once arises in discussing these questions from the fact that the viewpoint of the author is not made perfectly clear. One can readily conceive of lines of procedure under the reproduction method of computing value which might admit of opposite answers to all the questions mentioned. Under the circumstances, the dis-

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

Mr. rsted. cussion of these questions is likely to develop divergent opinions, depending on the viewpoint of the writer.

The reproduction method of valuation seems to require more hammering to reduce it to proper form and more clearly to restrict or fix the line of procedure; and, until this is done, there is little hope of arriving at a conclusion on questions of the kind raised by the author, especially when considered somewhat abstractly. In many instances computations relating to the rate of return on values found by the reproduction method have not sufficiently discriminated between a fair rate of return on the value of the depreciable and the non-depreciable property, on the value of the tangible and the intangible property, and on the value which is actually earned and that which includes an unearned increment. Such questions as these seem to have been overlooked in most of the rate cases of recent years. Moreover, most of the units going to make up the value of a public utility property possess no value except for the particular purpose for which the public utility is used, though other units possess an intrinsic value of their own, entirely apart from and independent of the public utility with which they are connected. Investment in units of one class is obviously more hazardous than in the other class, and there should be a corresponding difference in the rate of return to which the respective classes of property may be entitled. The questions raised by the author fall in a similar category, and unless they can be discussed thoroughly and comprehensively in connection with all other questions which enter into the computations of value by the reproduction method, or by any other method, no conclusive answer can be given to them.

The writer's discussion will be brief, and directed largely to simplifying, rather than complicating, the questions propounded for discussion.

Several valuations of water-works properties by commissions of experienced men, for the purpose, among other things, of rate-making, have been carefully analyzed and platted for the purpose of seeing how the methods of valuation therein used would work out in practice. However, only that part of one of the analyses which seems to apply to this paper, will be considered. The value of this particular property was computed by the commission in accordance with the reproduction method, as used in many of the late valuation cases, and allowance was made for depreciation as well as for enhancement of value and for unearned increments of value. Depreciation was computed on the sinking-fund basis by assuming a life for thirty or more units going to make up the physical property, with sinking-fund increments uniformly bearing interest at the rate of 3½% per annum, regardless of whether the unit under consideration was short-lived or long-lived.

Under the assumptions made in the writer's analysis, the composite property might or might not be assumed to have a definite life, but, for the purpose of a part at least of this discussion it is considered as possessing definite life—a life limit having been assumed for all the units of the depreciable property. Mr. Kiersted

On the diagram, Fig. 1, are platted two curves, each point in each curve being platted with reference to the average age and equivalent annual percentage rate of depreciation of the composite property. One, the full-line (upper) curve, is for convenience termed a definite-life curve, the other, the broken-line (lower) curve, is termed a perpetual-life curve. Average age is computed in the usual manner by the weighted method. The equivalent annual percentage rate of depreciation is found by dividing the total depreciation, as of the date under consideration, by the average age of the composite property, as of the same date. Whenever a break occurs in either of these curves it indicates that some one of the many units considered has reached the limit of its assumed life. It is assumed that throughout the whole period under consideration the total investment in the physical property remains the same. This assumption is met in the analysis by making two computations whenever any unit, going to make up the composite property, reaches its life limit. One computation consists of determining the average age and the total percentage depreciation of the composite property at the time when a unit reaches its life limit and is depreciated 100%; the other computation consists of determining the average age and total percentage depreciation as of the same year and date as the preceding computation, taking into consideration the new or substitute unit as of the same class and cost as the replaced unit. The new curve, starting with the substitute unit, continues unbroken until a second unit reaches its life limit, when another break occurs and two similar computations are made and a new curve started, and so on until all the units have been thus considered, making what is for convenience termed a cycle.

During a cycle of this particular property the number of units replaced by new ones of the same kind and cost are as follows: four units four times, eight units three times, nine units twice, and ten units once.

These curves serve no particular purpose except to illustrate the variation of the equivalent annual percentage rate of depreciation of the composite property during different periods of a cycle. They indicate a progressive and regular increase of the equivalent average annual percentage rate of depreciation until units begin to disappear, which is characteristic of the sinking-fund method of computing depreciation. The computations made in connection with the analysis lead to the following suggestions:

Mr. rsted. First, the definite-life (full-line) curve:

a.—The total percentage cost of replacements during a cycle is 132.5% of the total investment in the physical structures composing the composite property, equivalent to the amount of an annuity of 0.67% of the investment in physical structures, earning interest in a replacement fund at the rate of $3\frac{1}{2}$% per year during an average age period of 63 years.

b.—The value of the investment at the close of the cycle is the same as at the beginning of the cycle, the replacements having been made promptly as each unit reached the limit of its assumed life period.

c.—In order that the annuity of 0.67% of the investment in physical structures may amount to the cost of replacement during a cycle, the annuity and interest increments accruing thereon must remain undisturbed in a replacement fund during the entire cycle.

d.—If the owner of the property allows the replacement fund to remain intact, and uses new money to make replacements, then at the expiration of a cycle his property is worth 100% plus the monies in the replacement fund.

e.—If the replacement fund accumulates in the manner before stated and remains intact, and the owner makes no replacements with new money (assuming such a condition to be possible) then at the expiration of a cycle he has no property, but is the owner of a large portion at least of the monies in the replacement fund.

f.—If the owner of the property makes replacements with money taken from the replacement fund, as units need to be renewed, then such monies as may be taken from the replacement fund cease to draw interest as a part of that fund, but earn, as a part of the plant, their *pro rata* of the fair return due the owner and of the annuity going into the fund. The result is a deficit in the replacement fund when computed as per "*a*" and "*c*", of a large amount at the end of a cycle. In order to prevent such a deficit, and at the same time to permit monies to be taken from the replacement fund for renewals of plant as conditions demand, the annuity should be about 0.95% of the investment in physical structures.

g.—Throughout the entire cycle, computations of the fair return and of the annuity to the replacement fund should be based on 100% value of the investment.

h.—Replacements having been made to the property as needed, and the property thereby maintained at 100% value, there is no need of a sinking fund to return the original investment, because the property possesses the characteristics of perpetual life.

i.—Assume the owner to sell the property at some time during a cycle, the replacements having been made from monies accumulated in the replacement fund, then the property would be worth to the

Mr. Kiersted

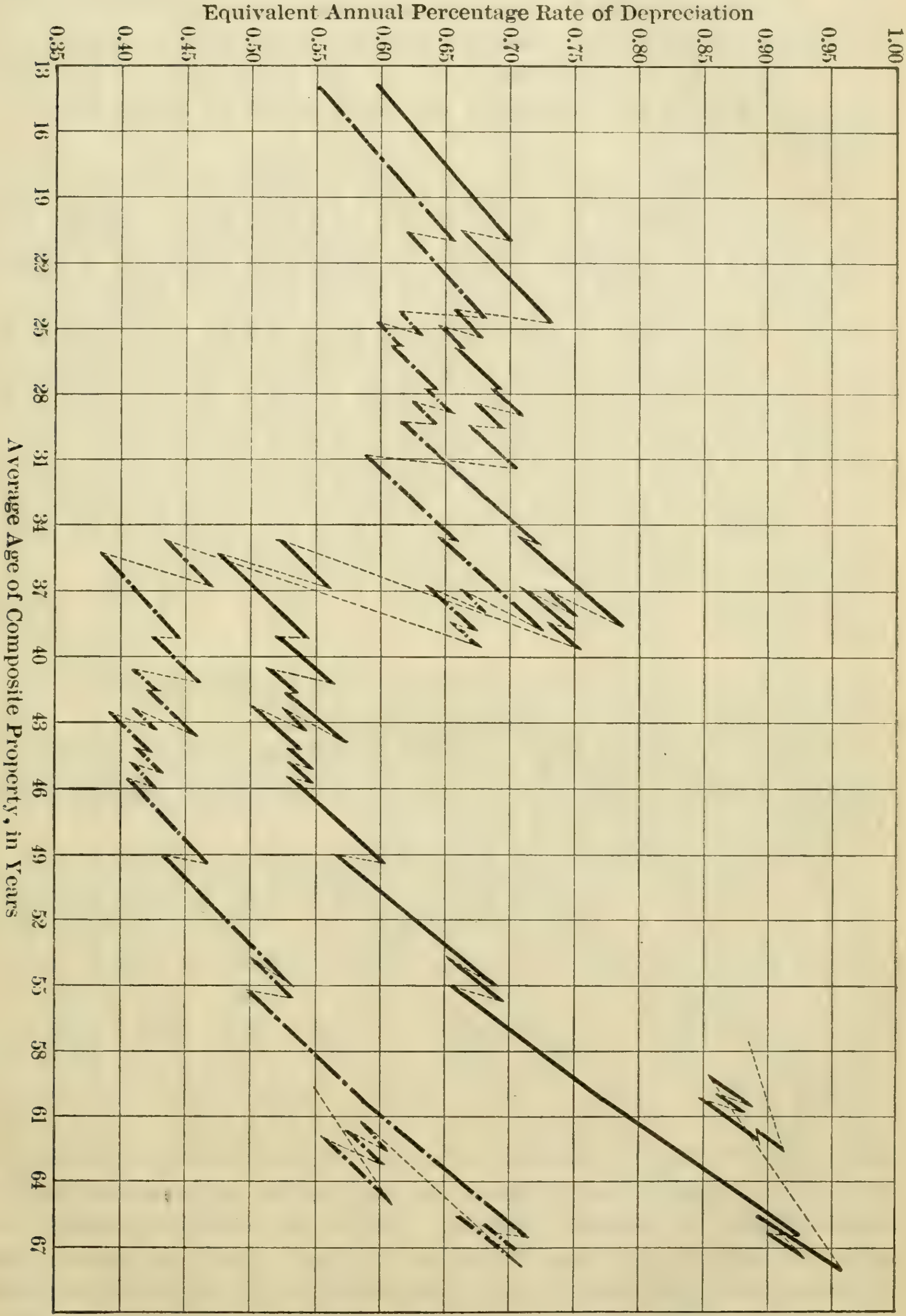

Fig. 1.

Mr. rsted. purchaser the full amount of the investment, provided the remaining monies in the fund go with the property. If the monies in the fund go to the owner (seller) then the purchase price should evidently be the original investment less the monies in the fund—equivalent to the investment less accrued depreciation. In any event, the purchaser must earn during his ownership on 100% value of the original investment.

Second, the broken-line (lower) curve is computed under assumptions precisely similar to those of the full-line curve, except that, on the theory of perpetual life of the composite property, a few of the long-lived units, such as cast-iron water pipe and reservoirs, are assumed to depreciate in part only; in other words, only partial replacement is necessary during a cycle for these particular units. On this theory, the total amount of renewals is 114% of the original investment, and, accordingly, the annuity to the replacement fund is found to be 0.80% of the original investment for an average age of 65 years, assuming replacements to be made from monies in the fund; or to be 0.53% of the original investment, if the fund remains intact and renewals or replacements are made with new money.

The general conclusions which can be drawn from this line of discussion are:

1.—That the position of Mr. Grunsky, in the paper alluded to, seems to be well taken, from the assumed point of view.

2.—That, in considering a public utility like a water-works, the composite property should be considered as possessing perpetual life, although the units may have definite life.

3.—That a replacement or renewal fund, properly proportioned, is the only protection that most public utilities need against deterioration; but, in order to cover contingencies like premature obsolescence, accident, and deterioration from unexpected causes, as well as deterioration resulting from ordinary wear and tear, the annuity to the fund should in all probability be not less than, and may even exceed, 1% of the value of the physical property of a water-works. Such a fund would embrace the replacement fund, and can be comprehensively termed a general maintenance fund. As a rule, no provision for a sinking fund to return the original investment need be made so long as the replacement fund is adequate and is properly applied. There are sufficient data, with respect to nearly every class of public utilities, to guide in determining an equivalent percentage rate of deterioration of physical structures by which to compute the annuity to a replacement fund. The annuity itself will doubtless vary for different properties of any given class. The annuity to a fund should be considered as earning interest at

Mr. Kierste

sinking-fund rates, the interest being considered, of course, as a part of the fund. None of the increments to a replacement fund should be capitalized. The monies in the fund should be used as needed for replacements, and should not be surrounded by a wall which prevents their proper application in that direction.

4.—That the question of whether a replacement fund shall or shall not earn a return is not one for consideration by a public utility commission in definite detail. The office of such a commission is to see that there is a fair return for rendering good service, and that the properties are maintained in such condition and so operated as to serve the public properly. The question whether the monies in a replacement fund should be considered as a part of the property for use only in replacements or to go to the owner of the property, is a debatable and important one. Should it go to the owner, he is obliged to credit the fund with the regular sinking-fund interest increments, but it cannot be conceived that any public utility commission would seek to destroy the incentive to good business management by ruling against the earning of a return on monies in such a fund, so long as replacements with full-value units are made promptly as needed; nor should such a commission delay in calling to account a management which ignores the public interest by delaying replacements when needed, in order to earn a return on the monies in a replacement fund. If the fund remains attached to the property and is to be used for replacements only, the annuity must be proportioned with some liberality, in order to avoid, if possible, a deficit; but if a deficit occurs and new money goes into replacements, then the owner is entitled to a re-rating of the property, a return of the value of his new-money replacements, or an increase of capital investment. On the other hand, the public is entitled to similar consideration under reverse conditions. In other words, properties must be re-rated periodically.

5.—That depreciation should be considered in connection with the valuation of the physical structures of public utilities on the reproduction basis. Depreciation may or may not be deducted from the value thus found, depending on the manner of the application of monies in the replacement fund and the extent to which the unearned increment enters into consideration. If replacements have been promptly made, and the policy of the management is to maintain high plant efficiency and good public service, there should be no depreciation of original cost value of existing physical property in a rate case; but, where reproduction forms the basis of value for rate-making, and such value includes an unearned increment, due to changed physical conditions or enhanced values of certain units resulting from city environment, and the same rate of return is allowed on these elements of value as on other portions of the property, then deprecia-

Mr. Oersted. tion should be allowed in computing fair return; but if the enhanced values and unearned increments of value were put in a class by themselves and were allowed such a rate of return as may be equitable, then depreciation of the physical property might be as consistently ignored in a rate case based on reproduction as when based on original cost value.

6.—That, in accounting, there is no necessity for dividing a property into groups of units according to some assumed life and charging each account with depreciation in accordance with some mathematical rule. The property should be considered as a whole, or in groups arranged by structural classification, deducting from the values thus recorded in the accounts the cost or value of any article or unit that is abandoned or replaced, and adding to the account the substitute article or unit. In doing so, proper discrimination should be drawn between minor renewals belonging properly to the operating account and replacements which actually affect the investment. Every engineer must concede that the place for working out details and for clarifying and simplifying any plan or specification is in the office, in order to render the work of the field parties as simple, clear-cut, and free from the danger of error as possible, and it can be conceived that similar principles should apply to accounting.

Although much more can be said on the questions under discussion, if considered in their relation with the general subject of valuation, and from a practical rather than an academic point of view, the writer is not inclined to extend the range of discussion beyond that laid down in the paper, except to add one or two remarks, particularly as the writer's discussion of Mr. Grunsky's paper presents relevant views which Mr. Alvord seems to indorse, in part, at least.

The writer has stated previously that the reproduction method of valuation needs more hammering to reduce it to proper form for general use. The attempt to apply rules to valuation work, with mathematical precision, is certainly a mistake, and leads the adherents of these rules into error in matters which necessarily involve the use of an enlightened and experienced judgment unfettered by precedent or rule. So many times the "yard stick" of value (to use the term of another) in contact with a mind overheated with the discovery of some element of value hitherto disregarded, or with some new idea, has been so expanded as to distort a line of reasoning believed at the time to proceed from an elevated or progressive point of view; or, on the other hand, the "yard stick," with the middle half cut out and the ends of the remaining portion joined with unaltered dimension at either end, has been used as though of up-to-standard length. There is no possible way of reconciling the differences thus encountered, or of bridging the wide gap exposed in the deductions thus made, except

by going back and standardizing the "yard stick." In other words, fundamental principles must be agreed on, and these principles must be applied consistently, intelligently, and cautiously, and with due consideration of the character, condition, and individuality of each property to be valued. In no two instances will the application of the principles be the same, but the results of the work of two or more appraisers should not vary so much as to be irreconcilable, or as to cause confusion of mind for those who hope to be profited by the expert's wisdom. Mr. Kiersted.

GEORGE B. STONE, ASSOC. M. AM. SOC. C. E. (by letter).—The question of depreciation of public utility corporations is of great moment at the present time to all members of the Profession, and especially to engineers engaged in the work of valuation of public service corporations, and is a subject of vital importance to both the public and the financial interests. Mr. Alvord's paper is of great interest, and he has presented his view of the question clearly and concisely. To the broad discussion that must take place before we finally come to an agreement, his paper is a valuable contribution. Mr. Stone.

The writer's experience has been entirely in railroad work, and in the remarks that follow he has confined himself to that point of view.

The valuation of common carriers, as outlined by the Federal law, gives a suggestion of "the cost of reproduction less depreciation" and the question immediately arises: "What shall this depreciation be, and how shall it be disposed of? The writer cannot agree with Mr. Alvord that depreciation is the same for all purposes of valuation, and will attempt to outline the reasons.

Depreciation for Rate-Making and Fair Return.—W. J. Wilgus, M. Am. Soc. C. E., in his paper, "Physical Valuation of Railroads,"* stated the question of depreciation for the purpose of rate-making very clearly and convincingly from the standpoint of the accountant working with an engineer on this subject, and the writer agrees absolutely with those views. The writer believes, however, that there are still other and equally important reasons why depreciation, as the word is commonly taken, should not be deducted from the cost of reproduction new, as of the date of the valuation. Stripped of all theories and technicalities, valuation for the purpose of rate-making becomes a question of allowable rates for a given service due to the corporation from its patrons. The Federal law suggests that these rates shall be determined on a basis of fair return based on the cost of reproduction new less depreciation, but it does not state how this subject of depreciation shall be considered, and has left unsettled a question which is open to much discussion and controversy. The object of the law was to furnish a means of settling many of the disputes that have arisen

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 203.

Mr. Stone. over the question of rates in the past, and to prevent and settle all such controversies in the future. Such being the case, the consideration of depreciation for rate purposes becomes one of service, and not physical value, unless the service of a given property has fallen below that which might reasonably be expected from a given investment. If it is found, in the valuation of a given property, that the service is less than might reasonably be expected from a given investment, then the property should be assessed a depreciation charge equal to the amount of depreciation from this source. This applies to a well-managed, going property, for if a property has been poorly managed, and there is evidence of needless expense and unwarranted waste, no valuation based on the cost of reproduction of its physical parts would ever give a correct value for rate-making or for any other purpose except that of taxation. Many properties will come between the extremes, but the principle remains the same.

Depreciation for Taxation.—The law, as practised, in regard to the taxation of all property, recognizes but one value: the amount that a given property will bring at forced sale; and, for taxation purposes, depreciation assumes an entirely different meaning from that for rate-making; in that it becomes a recognizable physical value, it becomes a problem of actual worth as shown from the inventoried cost new less any depreciation value due from any cause that affects its physical condition. It is a problem of actual physical worth, and cannot include anything but its true dollars-and-cents value. "A" owns a certain lot of land for which he paid $500 and can sell it for this amount at any time that he chooses, but, by waiting a little, he can sell it for $1 000. The $500 is the quick-sale or taxable value, and the larger figure is the sale value. This principle of taxation has been generally recognized by the Courts in taxation cases. The general principle of this subject is very simple, and the writer fails to find any reason for complicating it by the adoption of assumptions that do not exist in every-day practical railroad management; nor does he believe that any valuation based on theoretical assumptions will stand in the Courts. Depreciation for taxation can be the loss due to physical age of a property and nothing else, under the present construction of the tax laws.

Depreciation as Applied to Sales Value.—This phase of the subject is very closely allied to that of taxation, but has many features that prohibit the combination of the two under one head; it is also closely allied to depreciation for the purpose of rate-making, in that it has a value which is dependent on service. There are so many phases to this portion of the subject that the writer does not believe that, at this time, it would be either good business or good engineering to attempt to lay down a general law on the subject; nor, at the present stage of the matter, could a satisfactory general law be laid down. Every

carrier has a different problem to solve when it comes to the placing of a sale value on its property. The appraiser has the simplest portion of his work complete when he has finished the physical inventory of the property and made the proper deductions due to the physical depreciation of the property as of any certain date. He has many questions to ask himself: Is the location as it stands the best to be found, considering the traffic? In case the line were sold, would the purchaser rebuild any portion of it for economic reasons? Are there any competing lines built on a more economic basis which affect the earning power of the road under consideration? All these, and many others of a like nature, must be answered equitably before depreciation for sale value can be determined. Such questions as these have values of depreciation, and the appraiser must determine the true values and consider them a part of his appraisal. One road may not have cost double what its rival did, and yet the service of both roads is the same. There is a definite depreciation value here, chargeable against the more costly road, in case an appraiser was placing a sale value on it; but, another critic will say, there would be an appreciation value in favor of the cheaper road in case a sales value was to be placed on it. This is very true; there is a question of which value to apply, but that is for the appraiser to settle. The main point that the writer wishes to make is that there are values of depreciation for sales purposes which do not apply to either the taxation or the rate question. Mr. Stone.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.

Discussion.*

By Messrs. J. K. Finch, Alexander Potter, and Charles B. Buerger.

J. K. Finch, Jun. Am. Soc. C. E. (by letter).—In some cases reinforced concrete has been substituted for steel in the construction of tanks and stand-pipes, because it was believed it would result in economy, both in first cost and maintenance, and because such structures are generally on high ground, where they can be seen from afar, and it was hoped that a more satisfactory architectural effect could be secured by its use. The first of these expectations has generally been realized, but not the latter, not on account of the material, which is admirably suitable for architectural treatment, but because the failure to obtain a truly water-tight construction causes the marring of the surface of the structure by the formation of moist spots, due to "sweating," and patches of efflorescence. It has been pretty well established that the chief difficulties are: Mr. Finch.

1. Poor concrete;
2. Vertical cracks in concrete due to excessive stretching of steel under hoop tension;
3. Horizontal cracks between days' work, due to difficulty of joining old and new concrete, and the fact that concrete shrinks in setting and the successive layers are held apart by the vertical steel necessary for supporting the hooping rods; and
4. The formation of a horizontal crack, about 5 ft. above the base, in all cases where the bars in the floor or base have been turned up into the side-walls.

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*This discussion (of the paper by Edward Flad, M. Am. Soc. C. E., published in December, 1913, *Proceedings,* and presented at the meeting of February 4th, 1913), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Finch.

The remedy for the first is good materials and workmanship. A solution of the second difficulty has been proposed by H. B. Andrews, M. Am. Soc. C. E., in a paper entitled "A New Theory for the Design of Reinforced Concrete Reservoirs,"* presented before the Boston Society of Civil Engineers in 1910. This is the method followed by Mr. Flad in the design of the St. Louis tank. The writer desires to call attention to the fact that Mr. Flad, however, has used a working tensile strength for the 1 : 1½ : 3 limestone concrete of 290 lb. per sq. in., which appears to be excessive. It must be borne in mind, of course, that economy demands the use of a low factor of safety, say 1½, for the concrete, which is ample, as the ultimate safety of the structure depends on the steel, but this figure, 290 lb., has been taken apparently from Mr. Andrews' paper, in which he proposed this stress for a 1 : 1 : 2 concrete, not a 1 : 1½ : 3 mixture.

It is unfortunate that there are so few tests of concrete in tension on which to base a satisfactory working stress. The tests that have been made show a considerable range in values, due to the effect of different aggregates, methods of mixing, etc., and, generally, only unreinforced specimens have been tested. M. Considère made a number of experiments on reinforced concrete in tension, and concluded that the action of the reinforcing steel was to distribute the stress throughout the concrete and enable the latter to take a tensile loading considerably higher than the plain unreinforced material. It was shown later, however, that this was not proved by the tests, as M. Considère's method of computing the stress in the concrete and steel was incorrect. In fact, it was shown that a reinforced specimen developed cracks, and the concrete practically failed at about the same load as an unreinforced specimen. A few tests have been made at Columbia University on 6 by 6-in., 1 : 3 : 5 briquettes reinforced with 0.68 to 4.26% of steel, which showed a failure stress in the concrete, based on the break in the stress-strain curve of 205 lb. per sq. in., which is about the same as would be expected from an unreinforced specimen.

Mr. Andrews obtained his figure, 290 lb. for 1 : 1 : 2 concrete from a few tests he had made, which gave an average of 281 lb. per sq. in., by increasing his test value by 25 and 10%, which gave him 386 lb., which he considered as the ultimate strength for a large reinforced specimen, and that 290 lb. was sufficiently lower than this to be used in the design. The increase of 25% was to allow for "the usual increase in strength of large size over small specimens," his test pieces being 4 by 4 in. in cross-section; the further increase of 10% was to allow for the effect of reinforcing steel, and was probably based on M. Considère's deductions. Neither of these increases seems to be justified by any tests yet made, but Mr. Andrews' tests did give rather

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* *Journal*. Assoc. Eng. Soc., Vol. XLVI, 1911, p. 391.

low values. The following stresses are estimated to be fair values for the quality of concrete used in tank work: Mr. Finch.

Mix.	Age, in days.	Ultimate strength, in pounds per square inch.
1:1 :2	30	300-400
	90	400-500
1:1½:3	30	250-350
	90	350-450
1:2 :4	30	200-300
	90	300-400

The working stress should depend on the length of time to elapse before the tank will be filled, but 290 lb. per sq. in. appears to be high for a 1:1½:3 mixture.

There seems to be no remedy for the difficulty due to field joints, except pouring in one operation, which should be done whenever possible. Proper care in cleaning the surface of the old concrete, and the use of copper strips or dams, will doubtless do much to remedy this trouble.

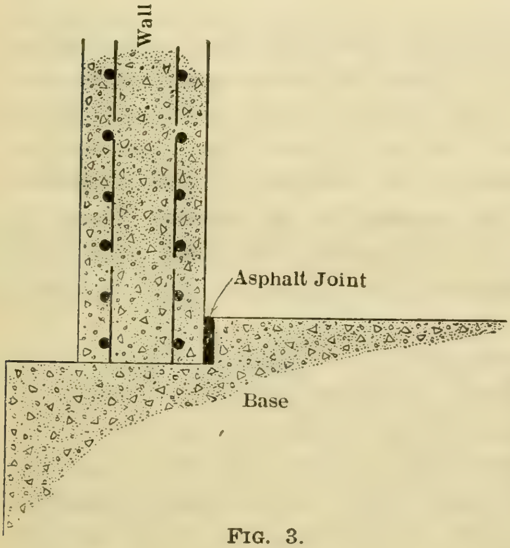

FIG. 3.

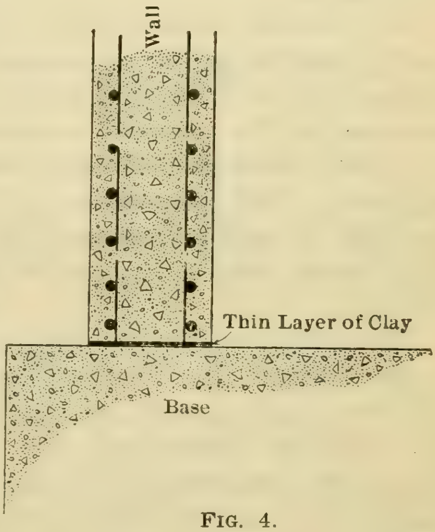

FIG. 4.

The formation of a horizontal crack, in cases where the bottom reinforcement has been run up into the sides, is due to the fact that the lower part of the side-walls of the tank is prevented from stretching under the hoop tension, due to the restraint of the bottom reinforcement. Indeed, the writer is of the opinion that in the lower part of Mr. Flad's design the wall acts as a cantilever in carrying the water pressure, and little or no hoop tension exists. He cannot see why this method of carrying up the bottom reinforcement has been followed in most cases. There is apparently no reason for not designing a tank simply as a vertical section of pipe resting on, but independent of, the base, as shown in Figs. 3 and 4. The pressure on the joint between

Mr. Finch. the sides and base should be investigated, of course, for the effect of wind and also for possible upward water pressure, but, in the majority of cases, this joint will always show an ample positive pressure.

The writer has used the form shown in Fig. 3 in preparing a design for his students in reinforced concrete, and has since learned that Mr. P. D. Johnson, Hydraulic Engineer of the Ontario Power Company, Niagara Falls, has made tests of the type of joint shown in Fig. 4, and has actually constructed the No. 2 Surge Tank of the Ontario Power Company on this principle. This tank is 70½ ft. high and 79 ft. in diameter, and no cracks have developed. The writer understands that a thin coating of clay was placed on the base under the walls before the wall concrete was deposited.

Mr. Potter. ALEXANDER POTTER, ASSOC. M. AM. SOC. C. E. (by letter).—As far as the writer knows this is the largest reinforced concrete reservoir in America of a type in which the hydrostatic pressure is resisted by ring tension; therefore it is of more than usual interest to the Profession.

In reference to reservoirs of this type, especially when approaching such a size, the writer has always felt that the importance of the connection of the cylindrical shell to the bottom is not always fully appreciated by the designer, especially as on it, more than, perhaps, on any other element of design, depends the success of the completed structure. The importance of this connection is evident from the fact that in such a reservoir, 150 ft. in diameter, the shell when filled with water expands so as to increase its diameter approximately 0.92 in.

Thus far, no attempts have been made to standardize this feature of the design. Two distinct methods, however, have been evolved in practice to take care of the expansion of the shell. In one, the bottom and shell are constructed as a monolith, and in the other the shell is entirely separated from the bottom by a water-tight expansion joint. The first of these methods is in common use, with more or less success, and is the one which seems to have been used successfully in constructing this settling tank.

Such a rigid connection, to be successful, calls for heavy steel reinforcement throughout the entire bottom, which, for economy, must be of as thin a section as possible so that, by stretching, it can readily adapt itself to the increase in diameter when the reservoir is full. This reinforced concrete bottom must be designed so that it can stretch in all directions on an average of 1 in 2 000 without developing cracks of sufficient extent to impair its water-tightness. There is every reason to believe that the elongation in the bottom is not distributed uniformly and that the bottom is stressed most in its outer portion and very little, if any, in its central portion. This unequal

Mr. Potter.

distribution would tend to give radial elongations in the outer circumference of the bottom considerably greater than the average value. It appears, however, that as long as the bottom is quite thin and is reinforced in all directions with at least 0.5% of steel possessing a high elastic limit, such a bottom can be maintained practically water-tight. Ordinarily, such a heavily reinforced thin bottom must be supported on a rigid and smooth foundation in such a way as to permit of as free expansion as possible. The bottom of the cylindrical shell must also be heavily reinforced to resist the stresses due to the bending moments set up in it by the restraining action of the bottom.

A rigid connection, such as used in the St. Louis reservoir, to be successful, is wasteful of structural material. It would appear to be more economical to use some type of water-tight expansion joint between the shell and the bottom, so as to permit the shell to expand irrespective of the bottom. This latter form of construction has already been used to some extent in circular tanks of relatively small diameter. A discussion on this point is invited, as the use of such expansion joints in tanks of large size, if proven successful, will tend to a more economic form of construction.

Another point on which there appears to be a wide difference of opinion is the proper thickness of the concrete section to be used in designing the shell. The author derives the formula, $T = \frac{pr - 9\,A}{12\,c}$ for the proper thickness of the concrete wall at any point, based on the assumption that vertical cracks are not likely to develop under a tension of 290 lb. per sq. in. in the concrete, and a ratio of 10 for the modulus of elasticity of steel to concrete. With the unit stresses assumed, the formula reduces to the simpler form that at any point the thickness of the concrete must be 42.7 times the area of the steel per inch of height of wall, thus limiting the steel reinforcement in the shell to 2.35 per cent.

The assumption on which the foregoing results are based appears to be an arbitrary one. There are any number of existing structures in which the ratio of steel to concrete exceeds 2.35%, and in which the concrete remains practically water-tight. Perhaps the most striking example can be found in cement-lined, wrought-iron pipe. In such pipe the steel reinforcement is frequently as high as 10%, and yet, under the high unit stresses to which the wrought-iron shell is often subjected in service, the cement lining possesses sufficient ductility to remain intact after years of service. Many sections of such pipe have been removed by the writer from distribution mains, the lining being found as perfect as on the day when laid. There are other structures, such as reinforced concrete pipe and stand-pipes, in which the ratio of steel to concrete in the shell exceeds 2.35%, which appear to maintain their water-tightness.

Mr. Potter. In a circular reservoir, the concrete in the shell performs two distinct functions: first, to render the structure water-tight; and second, to resist, in combination with the steel, all stresses due to bending moments which are likely to develop in the more or less rigid structure. The thickness of the concrete lining required for water-tightness, theoretically, is entirely dependent on the density of the concrete and the hydraulic head to which it will be subjected. Theoretically, therefore, it would be possible to use a very thin concrete section, provided the concrete was dense and water-proof, and the steel reinforcement very closely spaced. It is not found practicable, however, to do this. To build a truly circular reservoir is impossible. Owing to the lack of sufficient flexibility on the part of reinforced concrete, heavy bending moments are set up in the shell, due to irregularities of shape, and these must be resisted by the concrete in combination with the steel. It can readily be shown that, with only one row of circumferential reinforcing bars, the bending moments with their resultant stresses are very great and require a heavier concrete section than when two rows of circumferential reinforcing bars are used. The writer agrees with Mr. Flad that it is advisable to limit the percentage of steel reinforcement in the shell. The rule given by him, therefore, based on an apparently arbitrary assumption, appears to be conservative, especially when two rows of steel reinforcement are used. With only one row, it may be advisable to limit the ratio of steel to concrete to 1%, especially in reservoirs of large diameter.

As most engineers are, perhaps, more interested in the general subject of water purification than in the details of reinforced concrete design, the writer takes the liberty, without fear of unduly prolonging the discussion, to remark on the tank from the operating standpoint.

To remove the settled solid matter from this settling tank, it is necessary to draw off the water and place the tank temporarily out of commission. This is likely to happen at a time when the character of the water is such that it will need treatment most, and when the entire settling capacity of the water purification plant is most needed for efficiency. Unless such tanks are cleaned out regularly, the sludge deposits will greatly reduce their settling capacity, thereby lowering the efficiency of the settling process.

The writer has always been of the opinion that a tank in which the settled suspended and precipitated matter can be removed at intervals, without interfering with the operation or efficiency of the tank, possesses considerable advantages over one which must be emptied and put out of service to remove the deposited sludge. Such a basin operating continuously has a number of advantages over one which is operated intermittently. There is considerable saving in the

size of the settling basin when the settling process is carried on continuously. This saving may amount to as much as 50% over the intermittent installation in which two basins are used, decreasing somewhat with the number of basins. When the settling basin is operated continuously, its capacity is practically always available. This is not the case with the intermittent type. Mr. Potter.

There are a number of water purification plants in which the settling tanks are operated continuously. Without interfering with the operation and efficiency of the tank, the sludge is removed daily, and while it is still in a semi-liquid condition, before it has had time to compact. Settling tanks of this type have been used by the writer at McKeesport, Pa., and Georgetown, Ky. (both water-softening tanks), and also in the settling tank at Muskogee, Okla. In all these tanks, the sludge removal is accomplished by having the bottom of the settling compartments perforated with ½-in. to ¾-in. circular holes not more than 3 ft. from center to center in all directions. These holes discharge into an under-drain system leading to a common sump. Quick-opening valves are used to regulate the sludge discharge, and when such a system is well designed the possibility of the under-drains becoming clogged is very remote. All these plants are now in successful operation. The oldest is at McKeesport, and has been in continuous operation since 1907.

The cost of properly under-draining a settling tank so that is can be operated continuously is not excessive. For instance, the Muskogee settling basin,* constructed of reinforced concrete, 212 ft. square, 18 ft. deep, and holding 6 000 000 gal., was built for a contract price of $45 000, including all piping, valves, and under-drains.

Charles B. Buerger, Assoc. M. Am. Soc. C. E. (by letter).—The choice of a circular tension-ring type of tank, in this particular design, leads to the thought that there must be some economical dividing line such that tanks of smaller diameter are more economically built with a tension-ring wall, and those of larger diameter are most economical when built with cantilever walls. Mr. Buerger.

In actual practice the most economical style will depend largely on the particular details of design adopted, the unit stresses, the assumptions as to actions of the various parts, arrangement of metal, thickness of walls, the nature of foundation, and other varying data. There is, however, a general relation which can be established which shows what is for ordinary conditions the approximate maximum diameter for which the tension-ring type of tank is adapted.

Assume an ideal theoretical tank with tapered concrete walls of zero thickness at the top water level and a maximum thickness at the bottom, and assume that the active metal is in all cases 3% of the

* The details of construction and methods of operation are described in the *Proceedings* of the American Water Works Association, 1912.

Mr. erger. cross-section of the concrete, measured from the face of the concrete to the steel. Assume that the active steel is in all cases exactly equal to the theoretical quantity needed, with no allowance for laps or bonds; assume, also, that the steel at right angles to the active metal used to counteract shrinkage or temperature stresses, or to transmit stresses, is the same for the two types. For these conditions the only unequal element will then be the area or volume of the active steel.

Taking all dimensions in feet:

For the tension-ring type wall:

$$T = p\,r = 62.4\,hr, \text{ at any point } = f\,a \quad (1)$$

where a = the area of the metal,

and f = the unit stress in the metal.

Calling H the total height of the tank,

$$A\,f = \frac{62.4\,H^2\,r}{2} \quad (2)$$

and the volume of steel per foot of tank periphery is

$$V = \frac{31.2\,H^2\,r}{f} \quad (3)$$

For the cantilever-wall type:

As before, the total pressure acting on the wall above any plane

$$= 31.2\,h^2 \quad (4)$$

The moment of the pressure at that plane,

$$m = 31.2\,h^2 \times \frac{h}{3} = 10.7\,h^3 \quad (5)$$

For the assumption of 3% of metal and a ratio of modulus of elasticity of the steel to that of the concrete of 15, calling d the thickness of the wall to the steel

$$0.80\,f\,a\,d = m = 10.7\,h^3 \quad (6)$$

$$0.03\,d = a \quad (7)$$

$$a = \sqrt{\frac{0.40}{f}} \times h^{\frac{3}{2}} \quad (8)$$

$$d\,V = a\,dh = \frac{0.40}{f} \times h^{\frac{3}{2}}\,dh \quad (9)$$

$$V = \frac{2}{5}\sqrt{\frac{0.40}{f}} \times H^{\frac{5}{2}} \quad (10)$$

For the dividing line, at which the required steel is the same for both types of wall:

$$\frac{2}{5}\sqrt{\frac{0.40}{f}} \times H^{\frac{5}{2}} = \frac{31.2\,H^2\,r}{f} \quad (11)$$

$$r^2 = 0.000066\,f\,H \quad (12)$$

Taking $f = 144 \times 15\,000 = 2\,160\,000$

$$r^2 = 144\,H \quad (13)$$

$$r = 12\,\sqrt{H} \quad (14)$$

The general relation appears then that the maximum economical diameter of the tension-ring type of tank is 24 times the square root of the water depth. This relation will be modified in actual designs by several elements not considered in the foregoing. For instance, the quantity of steel to bond the cantilever wall into the floor of the tank has not been considered; and, as this is somewhat more in the cantilever type than in the tension-ring type, correction of this omission would be in favor of the tension-ring type. On the other hand, in the cantilever-wall type it is possible and economical to make the thickness of the wall greater than assumed in the foregoing equations, so that the percentage of metal is less than 3. The addition of this condition will favor the cantilever-wall type. Mr. Buerge

Applying this relation to Mr. Flad's executed reservoir, with the bottom of the wall at Elevation 97 and high-water line at Elevation 128, and a depth of 31 ft., it appears that the maximum economical diameter would be 134 ft. Mr. Flad's tank is somewhat larger than this, with a diameter of 150 ft.; but there are some conditions stated in his paper which amply explain and justify his choice of design. The foundation is stated to be river silt, and is no doubt ill adapted to support cantilever retaining walls satisfactorily. It is obvious that the more uniform loading given by the tension-ring type of tank justifies the use of additional steel reinforcement.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.

Discussion.*

By Leonard S. Smith, M. Am. Soc. C. E.

Leonard S. Smith, M. Am. Soc. C. E. (by letter).—This paper is a notable contribution to the literature of topographic surveying, and is of special interest to the writer because of its comparisons with the topographical work of the Barlow, United States and Mexican Boundary Survey.† In 1892-93 the writer made most of the actual stadia measurements on the tangent or boundary line from El Paso to Yuma, Ariz., besides sharing in the topographic traverses in the same region. In the 20 years which have elapsed since this work was done, he has given a good deal of thought to this subject in connection with both his instruction in topographic surveying at the University of Wisconsin, and also in certain stadia surveys of Wisconsin's most important lakes and rivers, some of which are described later. This work included the running of more than 2 000 miles of stadia traverses, and was secured under a great variety of climatic and other conditions, and for a variety of purposes. This experience has often illustrated the fact that there are many ways in which to make stadia measurements (as well as steel tape measurements), all depending on the character of the work and the accuracy required. Stadia measurements, in fact, should mean many things to all men, not one thing to some men. Mr. Smith.

* Continued from January, 1914, *Proceedings*.

† "Topography on the Survey of the Mexico-United States Boundary," by J. L. Van Ornum, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 259.

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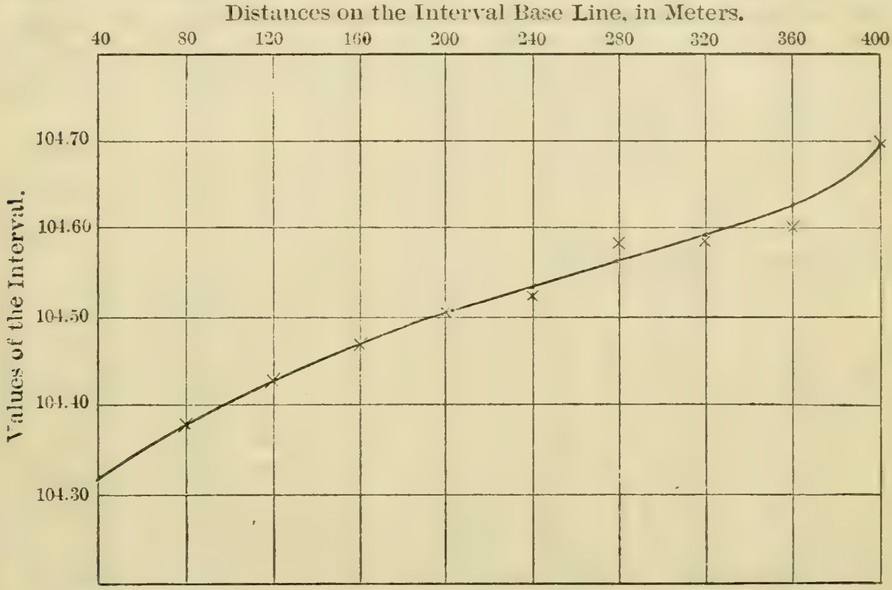
ith. The paper includes certain criticisms of the Barlow International Boundary work. The reader must not forget that this field work, as a matter of fact, was completed more than 20 years ago. In what line of engineering endeavor would it not be possible to criticize the work of two decades ago, in the light of present knowledge? It would be difficult to show that this work was not the most wisely planned topographic work of its day. The country covered included hundreds of miles of trackless deserts interspersed with scores of mountain ranges, all uninhabited and in most part devoid of water. In the Rio Grande boundary work, such extremes were not present, and, in general, conditions were much more favorable for accurate work.

The writer agrees thoroughly with Mr. Follett that adjustable stadia wires are "a delusion and a snare", also that rod levels for keeping the rod plumb are generally unnecessary, but he cannot agree with his statements regarding the general needlessness of an interval determination. It would be a very unimportant piece of work indeed on which the writer would use a steel tape without testing its length as reported by the dealer. The testing of the wire interval of a transit supplies the same information as the testing of the steel tape. So far as concerns the need of either test, it must be admitted that the wire interval is much more likely to change in the field than is the length of a steel tape. The author advocates sending the transit from the field to the manufacturer for readjustment of the interval whenever changes make this necessary.

The writer knows of engineers who send their transits to the instrument maker for even the ordinary adjustment, but he does not favor such a practice. When the adjustments and calibrations of a transit are made in the field by the engineer who uses such instrument, better results may generally be expected than if such adjustments and calibrations are made by some unknown mechanician in a distant city. Moreover, the danger of changes during transportation is always to be considered.

A careful examination of the fifty transits belonging to the Surveying Department of the University of Wisconsin, as well as a score of other instruments with which the writer at some time has been familiar, discloses the fact that only rarely has the stadia interval been found to be even approximately 100. In fact, even if the stadia wires were exactly one-one hundredth part of the focal length of the lens in the factory, one could not be certain that they would intercept 1 ft. on the rod for every 100 ft. of distance when used under vastly different circumstances in the field. This is not a matter of theory with the writer, but of frequent and accurate determination. He feels more certain to-day than he did 20 years ago that the general rule regarding the calibration of instruments applies equally well to the stadia interval; namely, that such calibrations, as far as con-

venient, should be made under the same circumstances as those to be met in the field work. Mr. Follett does not seem to think such precautions likely to result in more accurate work, and quotes certain stadia measurement made by the writer in 1892 to prove his point. It is now proposed to use these same stadia measurements, together with seventeen others, all checked by triangulation and steel tape measurements, to show that had not less but more care been taken in one important particular, a much higher accuracy would have resulted. Mr. Smith.



Characteristic Interval Curve, U.S.–Mexican Boundary Line, 1892–93.
Fig. 4.

In determining the stadia interval the method was to read the rods every 40 m. on a measured base line 400 m. long. A single determination of the interval was thus secured at each 40-m. point. These values are platted as the ordinates and the distances as abscissas of a curve, which, much generalized, is shown in Fig. 4. The adopted or working value of the interval was the average of all the ten values obtained at the various base distances. On account of the systematic differences in these partial intervals, this method would be correct only in case the stadia readings of the later field measurements were distributed approximately over the same range in distance as those of the interval determination. Now, in measuring the boundary line, rod readings at greater distances than 270 m. were not taken. The writer has reviewed the original field books concerned, and has found that had the computations for the stadia interval been restricted to the corresponding distance limit (270 m.), the resulting working interval would have been 104.44 instead of 104.60. This method was not followed at the time the measurements were made because its necessity then was not realized.

Mr. Smith. When preparing their report, 2 years later (1895), this feature of determining an interval was discussed and approved by the U. S. Commissioners. Table 25 is a record, which was checked by triangulation or steel tape measurements, of the writer's boundary stadia measurements and taken from page 235 of the U. S. Commissioner's report. A study of this table discloses the fact that the sign of the error of every stadia measurement checked is positive, that is, the distances, as measured by the stadia, were in every case too long, instead of being sometimes too short and sometimes too long, as would have been the case had the interval been free from large systematic error. The accumulated error of the 80 775 m. of stadia measurements was 94 m., or 1 in 858.

If these same measurements are computed by the use of $k = 104.44$ —the proper interval obtained, as explained previously—the resulting distances are shown in Column 7, giving the errors as stated in Column 8, and expressed fractionally in Column 9.

It will be noted that the errors of these measurements are of both signs, and that the total accumulated error of the 80 681 m. is only 28.8 m., or 1 in 2 800. It is also interesting to note that the two distances, Monument 84-85 and 85-86, measured by the writer (Nos. 8 and 9 of Table 25), and checked with steel tape by the author (Table 17), when computed by the proper interval factor (104.44), give results in remarkable agreement with the author's chained distances. Thus, the first measurement, Monument 84-85, checks exactly, and the stadia distance, Monument 85-86, checks with an error of only 0.9 m., or 1 in 4 930, instead of 1 in 657 and 756, as given in Table 17. The fact that the interval factor, 104.44, was computed from the actual field notes and published 15 years before the author's steel-tape measurements were made, should prove the entire reliability and independence of the writer's statements and comparisons. This work emphasizes the fact that an accurate stadia interval must be secured in order to prevent large systematic errors. The greatest advantage of stadia measurements—the certain approximate balancing of the accidental errors—is neutralized if large systematic errors are allowed to enter.

The measurements given in Table 25 are less than 10% of the boundary measurements made by the writer, and it was the general belief that they constituted the poorest work of the 800 000 m. In fact, they were checked over with the tape because of the large discrepancies from the Mexican measurements.

The writer shares the opinion, expressed in Mr. W. N. Brown's discussion, that the author under-estimates the accuracy with which stadia measurements may be made. Moreover, this greater accuracy can be secured without the expense of much labor or time. The office work of preparing an interval table* does not require more than an

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* Johnson and Smith's "Surveying," pp. 76-78.

hour, and with such a table the day's field notes can be reduced in 15 min. The time taken for an interval determination will be a small part of that involved in sending the transit to the instrument maker for adjustment of the wires. Mr. Smith.

TABLE 25.—ACCURACY OF STADIA MEASUREMENTS ON BARLOW, UNITED STATES–MEXICO BOUNDARY LINE, 1892–94.

(1) No.	(2) Between monuments.	(3) $k = 104.60$ U. S. stadia, in meters.	(4) Triangulation and steel tape, in meters.	(5) Error of stadia, in meters.	(6) Ratio of error.	(7) $k = 104.44$ U. S. stadia.	(8) Error of stadia, in meters.	(9) Ratio of error, 1 in.
1	43–44	3 691.74	3 690.14	+ 1.60	2 306	3 686.1	— 4.04	913
2	46–47	4 779.95	4 773.91	+ 6.04	790	4 772.7	— 1.21	3 944
3	48–49	4 445.93	4 443.17	+ 2.76	1 610	4 439.0	— 4.17	1 064
4	62–63	3 340.28	3 334.02	+ 6.26	532	3 335.2	+ 1.18	2 820
5	69–70	3 332.65	3 324.18	+ 8.47	392	3 327.5	+ 3.32	1 000
6	78–79	5 992.19	5 987.53	+ 4.66	1 280	5 983.0	— 4.53	1 320
7	83–84	6 067.67	6 060.42	+ 7.25	830	6 058.4	— 2.02	3 020
*8	84–85	3 983.66	3 977.6	+ 6.06	660	3 977.6	0.0	Infinity.
*9	85–86	4 449.98	4 444.1	+ 5.88	755	4 443.2	— 0.9	4 930
10	86–87	6 332.4	6 325.51	+ 6.89	920	6 322.9	— 2.61	2 420
11	87–88	6 698.31	6 685.18	+ 13.13	509	6 688.2	+ 3.0	2 230
12	91–92	5 698.71	5 693.35	+ 5.36	1 060	5 690.1	— 3.25	1 750
*13	92–93	4 465.2	4 462.58	+ 2.62	1 700	4 458.4	— 4.18	1 060
14	93–94	3 310.4	3 308.87	+ 1.53	2 160	3 305.4	— 3.47	924
15	94–95	3 478.41	3 475.07	+ 3.34	1 040	3 473.2	— 1.90	1 820
16	95–96	3 074.85	3 073.61	+ 1.21	2 540	3 070.2	— 3.41	900
17	96–97	3 627.78	3 623.94	+ 3.84	943	3 622.3	— 1.64	2 200
18	107–108	1 530.27	1 527.33	+ 2.94	510	1 528.0	+ 0.67	2 280
19	119–120	2 475.42	2 471.31	+ 4.11	600	2 471.7	+ 0.39	6 330
	Totals.	80 775.80	80 681.82	+ 94.0	858	80 653.1	— 28.8	2 800

* Stadia measurements referred to by the author in Table 17.

Greater accuracy in the stadia measurements may involve the saving of expensive control.

This is well illustrated by the methods adopted by the Wisconsin Water-Power Survey, a co-operative survey between the State and the U. S. Geological Survey.*

The specifications for this work were made nominally by the U. S. Geological Survey, but the writer was permitted to make several important changes which either added to the value of the work or greatly reduced its cost. One modification in the field work was the substitution of the magnetic needle for the control of all directions, the magnetic declination being frequently determined from Polaris observations. All distances were read by stadia and water levels; shore lines and contours were determined either by level or vertical-

* This work was done in 1906–07.

Mr. Smith. angle readings. Only every other turning point was occupied by the transit, a method which increased the speed by about 33 per cent.

Such horizontal control as was needed for the purposes of a preliminary survey was secured by checking on the section lines where they crossed the river, a feature which had the additional advantage of showing also the ownership of the riparian rights.

The vertical control was secured by running Wye-level lines and establishing permanent bench-marks on or near the river at intervals of about 2 miles, 400 miles of such levels being run at a total expense of $1 277. The unit cost of these levels varied from $2.16 per mile in a settled country with good roads to $4.85 per mile in a region devoid of roads and settlements. The instrument men on this work were junior civil engineering students at the University of Wisconsin.

As little on the subject of the accuracy of transit leveling has appeared in engineering literature, it may be of interest to examine in detail the accuracy of this work, which is shown in Tables 26, 27, and 28. In judging its accuracy, it should be remembered that the average turning-point distance was about 1 500 ft., and that the young men had had very little instrumental experience. The sensibility of the level corresponded to $\frac{1}{10}$ in. = 15″ of arc. The transits had object glasses $1\frac{1}{4}$ in. in diameter and the eye-piece magnified 24 times.

TABLE 26.—Accuracy of Transit Leveling on Black River, Wisconsin.

No. of check.	Distance, in miles.	Error in Stretch.				Accumulative Error.		Total.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	Miles.
1	2.3	0.09	....	+0.09	0.04	+0.09	0.04	2.3
2	2.3	0.44	....	+0.44	0.20	+0.53	0.12	4.6
3	2.4	0.17	0.69	−0.52	0.20	+0.01	0.001	7.0
4	3.6	0.23	0.54	−0.31	0.09	−0.30	0.029	10.6
5	4.6	....	0.14	−0.14	0.03	−0.44	0.030	15.2
6	5.2	1.50	....	+1.50	0.29	+1.06	0.050	20.4
7	3.8	....	0.57	−0.57	0.15	−0.49	0.020	24.2
8	3.2	1.01	....	+1.01	0.31	−1.50	0.055	27.4
9	3.6	0.72	0.90	−0.18	0.05	−1.32	0.042	31.0
10	3.6	0.20	....	+1.20	0.33	+2.52	0.073	34.6
11	6.3	0.49	0.59	−0.10	0.02	+2.42	0.06	40.9
12	4.8	0.31	0.75	−0.44	0.09	+1.98	0.044	45.7
13	5.2	0.27	0.17	+0.10	0.02	+2.08	0.041	50.9
14	1.8	....	0.78	−0.78	0.40	+1.30	0.025	52.7
15	5.3	....	0.79	−0.79	0.14	+0.51	0.009	58.0
Average, 0.15								

Table 26 shows that the largest error per mile on any stretch of the Black River Survey was 0.33 ft., and that the average error per mile was 0.15 ft. These errors, however, compensated to a remarkable extent, so that at the end of 10, 20, 31, 41, 51, and 58 miles the corresponding accumulative errors were only 0.03, 0.05, 0.04, 0.06, 0.04,

and 0.009 ft. per mile. The same engineer, Mr. Victor Reineking, next surveyed the Flambeau River with even better results, as will be seen from Table 27. Mr. Smith.

TABLE 27.—ACCURACY OF TRANSIT LEVELING ON FLAMBEAU RIVER.

No. of check.	Distance, in miles.	Error in Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error	Per mile.	Total.	Per mile.	
1	3.4	0.27	0.05	+ 0.22	0.06	+ 0.22	0.06	3.4
2	2.5	....	0.58	— 0.58	0.23	— 0.36	0.06	5·9
3	2.8	0.17	....	+ 0.17	0.07	— 0.19	0.02	8.7
4	2.5	....	0.28	— 0.28	0.11	— 0.47	0.042	11.2
5	1.5	....	0.08	— 0.08	0.05	— 0.45	0.045	12.7
6	2.2	....	0.10	— 0.10	0.045	— 0.65	0.043	14.9
7	4.1	....	0.69	— 0.69	0.16	— 0.134	0.070	19.0
8	4.4	0.62	....	+ 0.62	0.14	— 0.72	0.031	23.4
9	2.7	0.40	....	+ 0.40	0.14	— 0.32	0.012	26.1
10	1.3	0.20	....	+ 0.20	0.16	— 0.12	0.004	27.4
11	1.6	0.22	....	+ 0.22	0.13	+ 0.10	0.003	29.0
12	1.9	0.38	....	+ 0.38	0.20	+ 0.48	0.015	30.9
13	5.0	....	0.28	— 0.28	0.05	+ 0.20	0.006	35.9
14	2.5	....	0.68	— 0.68	0.15	— 0.48	0.013	38.4
15	1.2	0.34	....	+ 0.34	0.22	— 0.14	0.004	39.6
16	2.5	0.22	....	+ 0.22	0.08	+ 0.08	0.002	42.1
17	2.4	0.02	....	+ 0.02	0.008	+ 0.10	0.001	44.5
18	3.1	....	0.27	— 0.27	0.087	— 0.17	0.004	47.6
19	3.1	0.09	....	+ 0.09	0.028	— 0.08	0.002	50.7
20	4.2	....	0.02	— 0.02	0.004	— 0.10	0.002	54.9
21	1.0	....	0.06	— 0.06	0.06	— 0.16	0.006	55.9
22	1.8	....	0.27	— 0.27	0.15	— 0.43	0.008	57.7
23	3.6	0.51	....	+ 0.51	0.07	+ 0.08	0.001	61.3
24	6.6	....	0.03	— 0.03	0.005	— 0.05	0.001	67.9
25	4.8	0.46	....	+ 0.46	0.096	+ 0.51	0.007	72.7
26	1.4	0.11	....	+ 0.11	0.078	+ 0.62	0.008	74.1
27	4.0	0.05	....	+ 0.05	0.012	+ 0.67	0.008	78.1
28	3.2	0.22	....	+ 0.22	0.070	+ 0.89	0.011	81.3
29	3.6	....	0.56	— 0.56	0.153	+ 0.33	0.004	84.9
30	3.0	....	0.04	— 0.04	0.012	+ 0.29	0.003	87.9
31	4.4	....	0.03	— 0.03	0.007	+ 0.26	0.003	92.3

TABLE 28.—ACCURACY OF TRANSIT LEVELING ON WISCONSIN RIVER.

No. of check.	Distance, in miles.	Error of Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	18.1	1.14	1.77	— 0.63	0.039	— 0.63	0.039	18.1
2	14.6	....	2.73	— 2.73	0.187	— 3.36	0.103	32.7
3	15.0	0.45	....	+ 0.45	0.03	— 2.91	0.061	47.7
4	15.0	....	0.28	— 0.28	0.02	— 3.19	0.050	62.7
5	27.0	0.45	0.55	— 0.10	0.004	— 3.29	0.036	89.7
6	8.0	0.04	0.01	+ 0.03	0.003	— 3.26	0.033	97.7
7	16.3	0.16	0.53	— 0.37	0.023	— 3.63	0.032	114.0
8	6.3	0.10	0.04	+ 0.06	0.01	— 3.57	0.029	120.3
9	25.0	0.86	....	+ 0.86	0.03	— 2.71	0.019	145.3
10	23.0	0.85	0.17	+ 0.68	0.02	— 2.03	0.012	168.3
11	10.7	0.49	0.16	+ 0.33	0.03	— 1.70	0.010	179.0
12	6.0	0.54	....	+ 0.54	0.09	— 1.16	0.006	185.0
13	6.0	0.38	....	+ 0.38	0.06	— 0.78	0.004	191.0
14	9.0	0.34	0.13	+ 0.21	0.02	— 0.57	0.003	200.0

Mr. Smith.

TABLE 29.—ACCURACY OF TRANSIT LEVELING ON PESHTIGO RIVER.

No. of check.	Distance, in miles.	Error of Stretch.				Accumulative Error.		Total miles.
		Plus.	Minus.	Σ Error.	Per mile.	Total.	Per mile.	
1	2.1	0.16	....	+ 0.16	0.08	+ 0.16	0.08	2.1
2	1.9	....	0.17	+ 0.01	0.09	+ 0.01	0.002	4.0
3	3.6	0.72	....	+ 0.72	0.19	+ 0.71	0.09	7.6
4	1.5	0.13	....	+ 0.13	0.09	+ 0.84	0.09	9.1
5	4.7	0.32	....	+ 0.32	0.07	+ 1.16	0.08	13.8
6	2.5	0.13	....	+ 0.13	0.05	+ 1.29	0.078	16.3
7	5.0	0.02	....	+ 0.02	0.004	+ 1.31	0.062	21.3
8	5.0	0.16	....	+ 0.16	0.032	+ 1.47	0.056	26.3
9	7.6	0.17	....	+ 0.17	0.022	+ 1.64	0.049	33.9
10	4.0	0.0	0.94	— 0.94	0.23	+ 0.70	0.018	37.9
11	5.1	....	0.46	— 0.46	0.09	+ 0.24	0.005	43.0
12	2.4	....	0.04	— 0.04	0.017	+ 0.20	0.004	45.4
13	3.5	0.81	....	+ 0.81	0.23	+ 1.01	0.02	48.9
14	1.3	....	0.03	— 0.03	0.02	+ 0.98	0.02	50.2

The largest error was 0.22 ft. per mile, the average being 0.09 ft. per mile. Of the 31 checks, 14 show minus and 17 plus errors. This compensation, as shown in Table 27, is truly surprising. The total accumulative error at the end of the 92.3 miles was only 0.26 ft. or 0.027 ft. per mile.

Table 28 gives the results of transit leveling on the Wisconsin River Survey. Because of a misunderstanding of directions, the leveling on the first 32.7 miles of river was done by reading vertical angles, with the result that the largest errors are found in this stretch. In the remaining 168 miles of levels, the average error per mile is 0.04 ft., and the total accumulative error on the 200 miles is 0.57 ft.

TABLE 30.—SUMMARY OF TOTAL AND UNIT COSTS OF CO-

River surveyed.	Spirit Levels.								Topography.					
	Total miles.	Average per work day, in miles.	Lost time.		Miscellaneous.		Gross cost.		Total miles.	Average per work day.	Lost time.		Miscellaneous.	
			Days.	$	Days.	$	Total.	Per mile.			Days.	$	Days.	$
Wisconsin..........	1 873	4.9	4	35	4	35	403	2.16	197.0	2.5	16	142	15	135
Eau Claire.........	......	......	....	....	......	....	......	....	25.8	3.2	10	88	3	18
Peshtigo...........	40	5.0	7	59	5	38	194	4.87	81.7	3.3	1	12	13	182
Black..............	69	3.9	4	36	1	9	207	3.00	63.0	3.0	6	50	1	8
Flambeau..........	102	3.9	9	99	5	55	467	4.58	119.5	3.0	15	165	9	99
Totals and averages.	3 983	4.5	24	229	15	137	1 271	3.19	487.0	2.86	48	457	41	442

NOTE :—"Lost time" includes Sundays, holidays, and stormy weather. Miscellaneous

The 82 miles of transit levels run on the Peshtigo River were, for the most part, in rough, rocky, and heavily timbered country. The largest error per mile in any stretch was 0.23 ft., the smallest was 0.02 ft., with an average of 0.09 ft. The largest accumulated error was 1.64 ft., at a point 33 miles from the beginning. The accumulative error per mile gradually decreased from 0.09 ft. near the beginning to only 0.02 ft. at the end of the 82 miles. Mr. Smith.

It is perfectly manifest, from this record of 400 miles of transit levels, that for general preliminary surveys transit levels are amply sufficient without Wye-level control. Such omission would have reduced the cost of the survey by an average of $3.24 per mile.

Very careful records of the most important elements of cost data were kept, and are given in Tables 30 and 31. These elements, together with the costs of supervision, are assembled in Table 31. The greater cost of the Flambeau River work was due to working in the winter on the ice. The winter happened to be one of unusual severity, with very low temperatures and many deep snows. This resulted in the loss of about 30 work days, which increased the cost by $3.50 per mile of survey.

The surveys were mapped on a scale of 1 000 ft. to 1 in., and published by the U. S. Geological Survey on a scale of 2 000 ft. to 1 in. It will be noted that the total cost of 487 miles of river surveys, including both field and office expenses, was $4 798.30, or only $9.85 per mile for both profile and plan with land sections. This work furnishes a good example of the adaptability of the transit and stadia method.

OPERATIVE WATER-POWER SURVEY IN WISCONSIN, 1905-06.

		MAPPING.						TIME NOT SPENT IN FIELD WORK.						
Gross cost.		Net cost.		Gross cost.		Total per mile.		Lost time.		Miscellaneous.		Σ Time.		Σ Cost of field and office work.
Total.	Per mile.	Map.	Profile.	Map.	Profile.	Map.	Profile.	Days.	$	Days.	$	Days.	$	
981	4.97	120	11	160	16	0.81	0.08	22	182	25	210	47	392	$1 560 Dugan.
177	6.87	9	2	9	2	0.36	0.06	10	88	3	18	13	106	188 "
521	6.38	32	3	37	3	0.46	0.04	10	76	21	221	31	297	755 "
248	3.94	....	....	51	6	0.80	0.10	10	90	2	18	12	108	511 Reineking.
768	6.43	....	....	94	12	0.80	0.10	24	264	14	54	38	318	1 341 "
2 695	5.53	....	....	351	39	0.72	0.08	76	700	65	521	141	1 121	$4 355

time includes traveling and all causes not included in "lost time."

Mr. Smith.

TABLE 31.—TOTAL AND UNIT COST OF WISCONSIN CO-OPERATIVE RIVER SURVEY, 1905-06.

L. S. SMITH, Hydrographer in Charge.

River surveyed.	Spirit Leveling.			Topography.			Map and Profiles.			Cost of reconnaissance and supervision, per mile.	Total cost per mile of river.	Total cost.
	Total miles.	Total cost.	Cost per mile.	Total miles.	Total cost.	Cost per mile.	Total miles mapped.	Total cost.	Cost per mile.			
Wisconsin..	187.3	$403	$2.16	197.0	$981	$4.97	197.0	$171	$0.89	$0.73	$8.61	$1 698.00
Eau Claire.	0.0	0	0.00	25.8	177	6.87	25.8	11	0.44	1.00	8.29	213.80
Peshtigo....	40.0	194	4.85	81.7	521	6.36	81.7	40	0.50	0.70	9.98	815.20
Black.......	69.3	213	3.10	63.0	213	3.38	63.0	57	0.90	1.10	8.76	552.30
Flambeau..	102.0	467	4.58	119.5	796	6.69	119.5	106	0.90	1.25	12.71	1 519.00
Total.....	398.6	$1 277	$3.24	487.0	$2 688	$5.52	487.0	$385	$0.79	$0.90	$9.85	$4 798.30

Method—Transit and stadia with Wye-level control.
Scale—1 in. = 1 000 ft.
Contour interval = 10 ft. on land and 1 ft. on water.
Section lines located frequently.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

Sir WILLIAM HENRY WHITE, Hon. M. Am. Soc. C. E.*

DIED FEBRUARY 27TH, 1913.

William Henry White, the youngest child of Richard and Jane (Matthews) White, was born at Devonport, England, on February 2d, 1845. He attended the local schools until March, 1859, when, at the age of fourteen, he was apprenticed at the Royal Naval Dockyard. In addition to his practical work, he attended the Dockyard School, maintained by the Admiralty as a part of the technical training of its apprentices, where he showed his ability by winning an Admiralty Scholarship in 1863.

It was during this time that propulsion by steam was being substituted for sails, and sailing vessels were being reconstructed as screw-propellers. The use of iron in place of wood for the hulls of vessels was also being introduced, and young White had the advantage of combining practical work with the preparation of designs, the results of which were clearly shown in his highly responsible work of later years.

In 1864, the Admiralty, at the urgent request of the Institution of Naval Architects, established the Royal School of Naval Architecture at South Kensington, in connection with the Science and Art Department. This school, which was instituted in order that students in naval architecture might take advanced courses in that science, was afterward merged into the Royal Naval College at Greenwich. Sir William was one of the first students received at this newly founded college, taking first place at his entrance examinations in 1864, which standing he maintained during his three years of attendance. He was graduated in 1867, with the highest honors, as a Fellow (first-class). In 1870 he was made Professor of Naval Architecture at this school, which position he held until 1881. This work was in addition to his duties at the Admiralty. Among his pupils during this period were men who afterward became chief constructors and naval architects in the various navies of the world, and it was largely due to his influence that the school's high standing was acquired and maintained.

Immediately after his graduation from the Royal School of Naval Architecture, Sir William entered the Admiralty as Private Secretary to Sir Edward Reed, Chief Constructor, being engaged in the solution of scientific problems in naval architecture, etc. It was while he oc-

* Memoir prepared by the Secretary from information on file at the Society House.

cupied this position that he was brought directly in touch with the design of armored vessels which were then a new invention, thus adding to his experience as a designer and constructor.

In 1870, on the resignation of Sir Edward Reed and the appointment of a Commission to continue his work, of which Sir Nathaniel Barnaby was Temporary President, Sir William was made Professional Secretary to the Commission. In this position, and with the aid of his life-long friend, the late Mr. William John, he had charge of, and carried out, numerous experimental inquiries in regard to the stability of ships for the Commission appointed to investigate the capsizing of the ironclad *Captain,* in the Bay of Biscay. The results of these inquiries were embodied in a paper* on the subject presented before the Institution of Naval Architects, which greatly advanced the science of ship design.

In 1875, Sir William was promoted to the rank of Assistant Constructor, and, in 1881, he was advanced to that of Chief Constructor. While in this position he effected the organization of all the trained architects in the Admiralty into one corps—the Royal Corps of Naval Constructors—which organization has proved of great service to the British Navy. In 1883, when Sir William Armstrong was establishing his great shipyard at Elswick, England, he offered to Sir William the difficult feat of organizing and directing the Warship Building Department. While in this position he made designs for naval vessels for Austria, Italy, Japan, China, and Spain, and had charge of the construction of several vessels for the British Navy, among which was the *Victoria* which, in 1893, was sunk in the Mediterranean by a collision, with great loss of life.

In 1885, Sir Nathaniel Barnaby resigned his position at the Admiralty on account of ill-health, and on his recommendation, Sir William, having been released from his contract with Sir William Armstrong, Mitchell and Company, was appointed Director of Naval Construction and Assistant Controller of the Navy, which position he held until February, 1902, when he also was compelled to resign on account of ill-health.

Sir William began his work as Director of Naval Construction at the Admiralty at the beginning of a period of expansion in British Naval affairs. Prior to that time no uniformity in the vessels of the fleet had been attempted, and the reorganization of the administrative and operative departments was demanded. The Northbrooke programme, which comprised the carrying out of the construction of a number of new ships and the rapid completion of those already begun, was his first task, and the work was accomplished so expeditiously that when the Naval Defence Act of 1889 was passed, providing for

* "The Calculation of the Stability of Ships, and Some Matters of Interest Connected Therewith," by W. H. White and W. John, *Transactions*, Inst. of Naval Archts., Vol. XII, p. 77.

the construction of 70 ships at a cost of £22 000 000, sterling, Sir William had a chance to carry out his idea of homogeneity in a fleet, namely, ships bearing a distinct relation to each other and to the fleet as a whole. The Spencer programme of 1894 and the Goschen programme of 1896, the latter comprising the construction of the first dreadnought of the British Navy, were also carried out under his supervision. The development of the torpedo boat and the torpedo-boat destroyer was due to his skill, and he also designed the river gunboats built for the Nile Expedition under Lord Kitchener. When he retired in 1902, he had had responsible charge of the design and construction of 245 vessels, valued at about £100 000 000, sterling. All this vast work was done under constantly changing conditions of material, type, size, speed, armament, etc., and to him may be attributed the introduction of several innovations in British warships, notably that of water-tube boilers and the use of oil fuel for firing boilers. He had entered the Naval Service before the first real ironclad was ordered, and had seen the Navy develop from one of wooden ships to the present-day steel dreadnought.

He was awarded a C. B. in 1891 and a K. C. B. in 1895, and on his retirement from the Admiralty in 1902, Parliament voted him a special grant for "exceptional services to the Navy."

After a rest and the recovery of his health, Sir William began practice as a Consulting Naval Architect, and was engaged on many important works. He was a member of the Cunard Commission which decided the type of machinery for the *Lusitania* and the *Mauretania;* a Director of the firm of Swan, Hunter, and Wigham Richardson, Limited, the builders of the *Mauretania,* during her construction; a Director of the Parsons Marine Turbine Company; and a Director of the Grand Trunk Railway Company after that Company became the owner of steamships. He designed steamers with geared turbines for service between India and Ceylon, and was a Member of the Government Commission to investigate the question of load lines of merchant ships.

On February 27th, 1913, Sir William suffered a paralytic stroke at his offices in Westminster, and died the same day at Westminster Hospital to which he had been removed.

He was twice married, his first wife having been Miss Alice Martin who died in 1886. In 1890, he was married to Miss Annie Marshall who, with three sons, all of whom are officers in the British Navy, and one daughter, survives him.

Sir William was a frequent contributor to technical and engineering journals and to the publications of technical and scientific societies. He was also the author of several books, his "Manual of Naval Architecture" and "Treatise on Shipbuilding" having become classics, the former being translated into German, Italian, Russian, and Spanish.

His most notable contributions were made to the *Transactions* of the Institution of Naval Architects, the best known being the papers on the stability of ships (already referred to), the rolling of sailing ships, and the effect of bilge keels on rolling, and his description of the design of the battleship *Royal Sovereign*. That his versatility was not confined to engineering, but extended over a wide range of subjects, is shown by his contributions to various newspapers and magazines.

He was always greatly interested in questions of engineering and technical education, owing, perhaps, to the difficulties he experienced in acquiring his own training. He was Chairman of the Committee on the Education and Training of Engineers, appointed by the Institution of Civil Engineers in 1903, and was also a member of the Governing Body of the Imperial College of Science and Technology.

Sir William was connected with many engineering and scientific societies and associations, to which he was always ready to give advice and assistance, and many of these had honored him with offices of distinction. He was a Fellow of the Royal Society; Honorary Vice-President of the Institution of Naval Architects; Past-President of the Institutions of Civil Engineers, Mechanical Engineers, Marine Engineers, Junior Engineers, and the Institute of Metals; President-Elect of the British Association for the Advancement of Science, having been President of the Mechanical Science Section. He was also an Honorary Member of many other British and foreign technical societies, including the American Society of Mechanical Engineers and the Society of Naval Architects and Marine Engineers.

In 1911 he was awarded the John Fritz Medal by a Board of Award appointed by the four leading American engineering societies—Civil, Mining, Mechanical and Electrical—for "notable achievements in naval architecture." He had also received honorary degrees from many colleges and universities, among which were LL.D. from Glasgow, D. Sc. from Cambridge, Durham, and Columbia (New York City) Universities, and D. Eng. from Sheffield. He also belonged to the Athenæum and British Empire Clubs.

In regard to his personal characteristics, the following* was written by one who had been on terms of close friendship with him for more than 30 years:

"He was beyond compare the straightest and most generous man alive. He was, without exception, the most remarkable man I have known, and as honest as the day. From the start he had never a soul to help him, but achieved distinction by his own ability and industry. A foreign government made overtures to him to superintend the total reconstruction of its then somewhat infirm war flotilla. White, with that fine sense of honour which was inseparable from the man, re-

*The Times (London), February 28th, 1913.

fused a most lucrative offer because he considered the value he could return for the great monetary payment held out to him, could only have been acquired by his intimate knowledge gained in the service of our Admiralty. The amount of work that he did without repayment in those educational institutions which are bringing forward the most intelligent men of the new generation, was beyond compare. His self-negation, accurate knowledge, and charming manner extraordinarily enhanced by a great gift of description, made him such a professor of scientific and applied mechanics as the world has seldom seen."

Sir William Henry White was elected an Honorary Member of the American Society of Civil Engineers on December 16th, 1904.

HORACE GUY MERRICK, Assoc. M. Am. Soc. C. E.*

Died October 30th, 1913.

Horace Guy Merrick was born at Libertyville, Ill., on January 29th, 1879. His family moved to Manistee, Mich., when he was quite young.

Mr. Merrick's education began in the public schools at Manistee, and he was graduated from the High School of that place in 1898. During the next three years he was engaged on various public works in Michigan. In 1901 he entered the University of Michigan, from which institution he was graduated in 1905, with the degree of B. S. in Civil Engineering.

After his graduation he was employed by the United States Government as Junior Engineer on the survey of the Great Lakes until May, 1907, when he was transferred to the work of improving the Upper Mississippi River, with headquarters at La Crosse, Wis. He remained on this work until his death.

In 1913, Mr. Merrick took the examination before the United States Civil Service Board for registration and promotion to Assistant Engineer, and passed with much credit.

He was an energetic and capable civil engineer, and was held in high esteem by his superior officers. He felt no wish to become conspicuous, professionally or otherwise, shrinking from notice rather than courting it. He always under-estimated his own ability, and worked zealously, intelligently, and successfully from love of his Profession.

Mr. Merrick was elected an Associate Member of the American Society of Civil Engineers on May 7th, 1913. He was also a member of the Wisconsin Society of Civil Engineers.

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* Memoir prepared by W. A. Thompson, M. Am. Soc. C. E.

PAPERS IN THIS NUMBER

"AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING." JOHN C. KOCH. (To be presented Mar. 4th, 1914.)

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OF THE

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(INSTITUTED 1852)

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American Society of Civil Engineers

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

February 18th, 1914.—The meeting was called to order at 8.30 P. M.; T. Kennard Thomson, Director, in the chair; Chas. Warren Hunt, Secretary; and present, also, 129 members and 15 guests.

A paper by Harold A. Rands, Assoc. M. Am. Soc. C. E., entitled "Grouted Cut-Off for the Estacada Dam", was presented by H. V. Schreiber, M. Am. Soc. C. E., and illustrated with lantern slides. Mr. Schreiber also presented communications on the subject from Messrs. S. Howard Rippey and S. C. Hulse. The Secretary read discussions by Messrs. William H. Cushman and H. L. Coburn. The paper was discussed orally by Messrs. Lazarus White, H. V. Schreiber, Wilson Fitch Smith, and V. H. Hewes.

A paper by A. L. Harris, Assoc. M. Am. Soc. C. E., entitled "Diversion of Irrigating Water from Arizona Streams", was presented in abstract by the Secretary.

The Secretary announced the following death:

David Neilson Melvin, of Linoleumville, N. Y., elected Member, July 3d, 1878; died January 27th, 1914.

Adjourned.

March 4th, 1914.—The meeting was called to order at 8.30 P. M.; President Hunter McDonald in the chair; Chas. Warren Hunt, Secretary; and present, also, 164 members and 11 guests.

The minutes of the Annual Meeting, January 21st, and of the meeting of February 4th, 1914, were approved as printed in *Proceedings* for February, 1914.

The President announced that the Board of Direction, realizing that the canvass of the ballots on the proposed amendments to the Constitution would require a good deal of time, had appointed a committee of its own members for that purpose.

The proposed amendments are as follows:

AMENDMENT "A"

Amend Article VII—Nomination and Election of Officers—as follows:

Strike out Section 1, and substitute the following:

"The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year."

Strike out the first paragraph of Section 2, and substitute the following:

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, shall be appointed by the meeting to serve for two years. They shall be selected so as to provide, with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve Districts; and these, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society."

Strike out the word "and" in the fifteenth line of the third paragraph of Section 2, and after the figure "7" add: "8, 9, 10, 11, 12, and 13."

AMENDMENT "B"

Amend Article VII—Nomination and Election of Officers—as follows:

Strike out Section 1, and substitute the following:

"1.—The Board of Direction shall, from time to time, divide the territory occupied by the membership into thirteen geographical districts, to be designated by numbers. District No. 1 shall be the territory within fifty miles of the Post Office in the City of New York. Each of the other districts shall be, as nearly as practicable, contiguous territory, and shall be designated as Districts Nos. 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, and 13. The Board shall announce such division to the Society on or before the first day of March in each year."

Section 2.—Strike out Paragraphs 1 and 2, and substitute the following:

"2.—Seven corporate members, not officers of the Society, shall be elected annually by the members in the respective districts to serve for two years. They shall be elected so as to provide with the seven members holding over, two members from District No. 1, and one from each of the remaining twelve districts and these, with the five living last past Presidents of the Society, shall be a committee to nominate officers for the Society.

The manner of election shall be as follows:

Directly after the first day of October of each year, there shall be mailed to each corporate member in each district where an election is to be held a notice with form and envelope for voting, requesting him to suggest the name of one corporate member (not an officer of the Society) from his district as a candidate for nomination for member of the nominating committee; these suggestions to be made to the Board of Direction by a preliminary letter-ballot to be received by the Board at a meeting to be held about the first of November, and opened at said meeting, and counted under the direction of the Board. The polls shall be closed at noon on the day of said meeting and the time of closing the polls shall be stated in said notice. At least thirty days before the annual meeting there shall be mailed to every corporate member, a second notice with form for voting, said notice to state the time of closing of the polls on the final ballot, and to contain as nominees for the nominating committee from each voting district the names and residences of the two persons receiving the highest number of votes in each district, and such others as may have received the same number of votes as one of these in order of their standing, and the number of votes cast for each as the result of the preliminary ballot. On the final ballot the polls shall close at noon on the day preceding the annual meeting, and the ballots shall be canvassed under the direction of the Board. The members receiving the highest number of votes in their respective districts on the final ballot shall be declared elected members of the nominating committee. Announcement of elections by this ballot shall be made

at the annual meeting. In case of a tie vote in any district the annual meeting shall elect the member of the nominating committee from among the persons so tied. Vacancies in the Nominating Committee may be filled by the Board of Direction."

Paragraphs 3 and 4 of Section 2 of Article VII shall constitute a new section numbered 3, and for the words "This Committee" in the first line of Paragraph 3 substitute the words "The Nominating Committee"; and in line fifteen of the same paragraph strike out the word "and," and after the figure "7" add: "8, 9, 10, 11, 12, and 13."

Sections 3, 4, 5, 6, and 7 of Article VII shall be numbered Sections 4, 5, 6, 7, and 8, respectively.

In the second sentence of present Section 5 (new Section 6) of Article VII, insert after the word "nominated" the words "for Officers of the Society," so the said sentence shall read "This ballot shall include the names and residences of all persons nominated for Officers of the Society in accordance with this Article, their grades of membership, and, in case of nominees for Directors, the number of the district in which they reside."

AMENDMENT "C"

Amend Articles V and VI as follows:

Article V, Section 1.—Strike out the first sentence and substitute the following:

"The officers of the Society shall be a President, four Vice-Presidents, eighteen Directors, a Secretary, and a Treasurer. These officers, except the Secretary, together with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction in which the government of the Society shall be vested, and who shall be the Trustees as provided for by the laws under which the Society is organized."

Article V, Section 2.—First Paragraph. In first sentence strike out, "Secretary", so that the sentence shall read as follows:

"The terms of office of the President and Treasurer shall be one year; of the Vice-Presidents, two years; and of the Directors, three years."

Second Paragraph. In first line insert after the word "officer" the words "elected by the Society" so that the paragraph shall read:

"The term of each officer elected by the Society shall begin at the close of the Annual Meeting at which such officer is elected, and shall continue for the period above named or until a successor is duly elected."

Article VI, Section 4.—Strike out the first two paragraphs of the section and substitute therefor the following:

"The Secretary shall be a Corporate Member of the Society. He shall be elected annually by letter-ballot of the Board of Direction, which shall be ordered not less than 20 days after the Annual Meeting.

The ballots shall be returned to tellers, appointed by the President and canvassed at a meeting of the Board. A majority of the whole Board shall be required to elect.

The Secretary shall hold office until his successor is elected, shall devote his whole time to his duties, shall be under the direction of the President and the Board of Direction, and shall be the executive officer of the Society."

Article VI, Section 6.—Strike out this Section and substitute therefor the following:

"6.—The Secretary and Treasurer shall be paid salaries to be determined by a majority of the Board of Direction, this vote to be taken by letter-ballot to be canvassed by the Board of Direction. All other salaries shall be fixed from time to time, by the Board of Direction."

The Secretary presented the report of the Tellers, as follows:

The undersigned Tellers, having canvassed the ballots on the amendments to the Constitution, report as follows:

Total number of ballots received	3 203
Ballots from those not entitled to vote, or otherwise defective	26
Total number of ballots canvassed	3 177
Amendment "A"	
Number of ballots in favor	1 494
Number of ballots opposed	1 628
Number of ballots necessary to carry amendment	2 081
Amendment "B"	
Number of ballots in favor	1 550
Number of ballots opposed	1 612
Number of ballots necessary to carry amendment	2 108
Amendment "C"	
Number of ballots in favor	1 343
Number of ballots opposed	1 828
Number of ballots necessary to carry amendment	2 114

HUNTER McDONALD,
HENRY W. HODGE,
ARTHUR S. TUTTLE,
J. A. OCKERSON,
MORDECAI T. ENDICOTT,
GARDNER S. WILLIAMS,
J. WALDO SMITH,
CHAS. WARREN HUNT,
E. GERBER.

MARCH 4TH, 1914.

The President declared that all the amendments were lost, not having received two-thirds of all ballots cast.

A paper by John C. Koch, Assoc. M. Am. Soc. C. E., entitled "An Investigation of Sand-Clay Mixtures for Road Surfacing" was presented in abstract by the Secretary, and the subject was discussed by Messrs. Arthur H. Blanchard, James Owen, and Spencer J. Stewart.

H. deB. Parsons, M. Am. Soc. C. E., addressed the meeting briefly on the subject of the Main Drainage of New York City.

The Secretary announced the election of the following candidates on March 4th, 1914:

As Members

Martin Joachimson, New York City
John George Gale Kerry, Toronto, Ont., Canada
Robert Edgar Milligan, New York City
Arthur Alvord Stiles, Austin, Tex.
Charles Mason Talbert, St. Louis, Mo.
Harry True Welty, New York City

As Associate Members

Frederick Wilhelm Albert, Washington, D. C.
John Joseph Baker, Mineral Point, Pa.
Earl Huntington Barber, Boston, Mass.
Orville Berton Carlisle, Chicago, Ill.
William Henry Evers, Cleveland, Ohio
Trygve Daniel Bodtker Groner, Salt Lake City, Utah
Cecil Shields Haig, Fortaleza, Ceara, Brazil
Andrew Pearson Hoover, Paterson, N. J.
Edward Horton Jones, Clifton, Ariz.
Sargent Felix Jones, Edmonton, Alta., Canada
Walter John Kackley, Miami, Fla.
John Laylin, Columbus, Ohio
Gustav Jaeger Requardt, Baltimore, Md.
Daniel Bell Sayer, Albany, N. Y.
Howard French Schryver, Columbus, Ohio
Stephen Elliott Shoup, Cincinnati, Ohio
Maurice Joseph Sullivan, Houston, Tex.
Henry Casper Tammen, Kansas City, Mo.
Henry Jackson Tippet, Vancouver, B. C., Canada
Richard John Wulzen, Juneau, Alaska

As Juniors

Harry Arthur Armstrong, Sacramento, Cal.
John Goodin Bailhache, San Francisco, Cal.
Jacob Bendel, New York City

CLARENCE WINSTON COOPER, Wilson, N. C.
WALLACE DUNCAN DU PRE, Spartanburg, S. C.
DAVID ARTHUR HEDLUND, Spokane, Wash.
THEODORE SCHUYLER HERSEY, San Francisco, Cal.
WILLIAM HOLDEN, Fort Worth, Tex.
WILLIAM HUGO JAENICKE, San Francisco, Cal.
EMORY WILSON LANE, Ithaca, N. Y.
JUAN LORIA MATAMOROS, Washington, D. C.
LEONARD KYRAN MOYLAN, Troy, N. Y.
GEORGE WASHINGTON RAPELLI-OLIVER, Buenos Aires, Argentine Republic
JOHN WAGNER, JR., Phœnixville, Pa.

The Secretary announced the transfer of the following candidates on March 4th, 1914:

FROM ASSOCIATE MEMBER TO MEMBER

LAURENCE ADAMS BALL, New York City
SHIRLEY CLARK HULSE, Bedford, Pa.
RAY EMERSON KOON, Portland, Ore.
CHESTER ALEXANDER SMITH, Kansas City, Mo.
THEODORE NELSON SPENCER, Philadelphia, Pa.

FROM JUNIOR TO ASSOCIATE MEMBER

GEORGE SIMPSON ARMSTRONG, London, England
ROY CECIL GRAY, Kansas City, Mo.
TOM HIND HUDSON HARROD, Parr, S. C.
CYRUS PIERCE HOWES, São Paulo, Brazil
KENNETH HOWARD OSBORN, Cleveland, Ohio
CLIFFORD FRENCH PHILLIPS, St. Louis, Mo.
HENRY SLICER REGESTER, JR., Baltimore, Md.

The Secretary announced the following death:

WEBSTER GAZLAY, of Louisville, Ky., elected Member, June 7th, 1905; died February 17th, 1914.

Adjourned.

SPECIAL MEETING

March 11th, 1914.—The meeting was called to order at 8. 30 P. M.; Vice-President J. Waldo Smith, in the chair; Chas. Warren Hunt, Secretary; and present, also, 238 members and 37 guests.

The Chairman announced that the meeting had been called for the purpose of discussing the Progress Report of the Special Committee on the Valuation of Public Utilities.

Frederic P. Stearns, Past-President, Am. Soc. C. E., Chairman of the Committee, opened the discussion, illustrating his remarks with lantern slides. The subject was discussed also by Messrs. Alexander C. Humphreys, F. Lavis, H. C. Phillips, W. W. Crehore, P. W. Henry, Morris Knowles, A. G. Nicolaysen, F. B. Maltbie, and Henry Floy.

On motion, duly seconded, the meeting was adjourned until a date to be fixed later by the Board of Direction.

(Since adjournment of the meeting the date for continuing this discussion has been fixed for Thursday, April 2d, 1914, at 2.30 P. M. If necessary, the discussion will be continued at 8.30 P. M.)

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

April 1st, 1914.—8.30 P. M.—A regular business meeting will be held, and a paper by F. Lavis, M. Am. Soc. C. E., entitled "The Gauge of Railways, with Particular Reference to Those of Southern South America", will be presented for discussion.

This paper is printed in this number of *Proceedings.*

April 2d, 1914.—2.30 P. M.—A Special Meeting for continuing the discussion on the Progress Report of the Special Committee on the Valuation of Public Utilities, has been fixed for Thursday, April 2d, 1914, at 2.30 P. M. If necessary the discussion will be continued at 8.30 P. M.

April 15th, 1914.—8.30 P. M.—At this meeting a paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled "The Possibilities in Bridge Construction by the Use of High-Alloy Steels", will be presented for discussion.

This paper is printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-sixth Annual Convention of the Society will be held at Baltimore, Md., from June 2d to 5th, 1914, inclusive.

The following Committees to take charge of arrangements have been appointed:

Committee of the Board of Direction:

James H. Edwards,
George W. Fuller, Chas. Warren Hunt.

Local Committee of Arrangements:

Francis Lee Stuart, *Chairman.*

Mendes Cohen, W. Anderson Polk,
W. W. Crosby, Layton F. Smith,
J. E. Greiner, H. A. Warren,
F. H. Hambleton, E. B. Whitman.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

THE J. JAMES R. CROES MEDAL

This new medal has just been awarded for the first time. The rules for its award will be found in the List of Members. For the information of members photographs of it are here reproduced. The medal is of fine gold and so arranged that it can be made useful as a watch fob.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meeting)

December 19th, 1913.—The Annual Meeting was called to order; President Wing in the chair; E. T. Thurston, Jr., Secretary; and present, also, 74 members and guests.

The Reports of the Secretary and Treasurer, respectively, were read, and the following officers were elected:

President, C. H. Snyder,
Vice-President, H. L. Haehl.

A paper by Mr. J. B. Pope, Assistant Engineer of the Southern Pacific Company, on "The Valuation and Original Cost of Railroads", was presented by the author, and the subject was discussed by Messrs. C. E. Grunsky and F. T. Robson.

Adjourned.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., and, until further notice, will take place at the lunch room of the Denver Dry Goods Company.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

February 14th, 1914.—The meeting was called to order; President Ridgway in the chair; E. F. Vincent, Vice-President, acting as Secretary; and present, also, 20 members and guests.

The minutes of the meeting of January 9th, 1914, were read and approved.

An emblem for use on stationery, etc., was adopted by the Association, consisting of the shield of the American Society of Civil Engineers, surrounded by a shield-shaped band bearing the words "Colorado Association of Members".

The proposed Amendments to the Constitution of the Society were discussed.

A paper by John E. Field, M. Am. Soc. C. E., on "Interstate Water Rights", was presented by the author, and was followed by a general discussion of the subject and of International Water Rights.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club.

At the meeting of the Association on December 29th, 1913, the new Chairman, John Ruddle, M. Am. Soc. C. E., was installed, and Messrs. Park A. Dallis and G. R. Solomon were appointed members of the Executive Committee. T. P. Branch, Assoc. M. Am. Soc. C. E., was elected Secretary.

Philadelphia Association

On December 22d, 1913, the Philadelphia Association of Members of the American Society of Civil Engineers was organized, with the following officers: George S. Webster, President; Richard L. Humphrey and F. Herbert Snow, Vice-Presidents; John Sterling Deans, J. W. Ledoux, Edgar Marburg, and H. S. Smith, Directors; S. M. Swaab, Treasurer; and W. L. Stevenson, Secretary. The meetings of the Association will be held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year: Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

Southern California Association

On January 5th, 1914, the Southern California Association of Members of the American Society of Civil Engineers held its first meeting at the University Club, Los Angeles, Cal., and elected the following officers: J. B. Lippincott, President; Charles T. Leeds, First Vice-President; George S. Binckley, Second Vice-President; W. K. Barnard, Secretary; and Charles H. Lee, Treasurer.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, C. S. MacCalla; Vice-President, U. B. Hough; Second Vice-President, Morton Macartney; Secretary-Treasurer, A. D. Butler.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 219 Market Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, Room 6, City Bank and Trust Company Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From February 3d to March 2d, 1914)

DONATIONS*

A TREATISE ON THE INSPECTION OF CONCRETE CONSTRUCTION :

Containing Practical Hints for Concrete Inspectors, Superintendents, and Others Engaged in the Construction of Public and Private Works. By Jerome Cochran, Jun. Am. Soc. C. E. Cloth, 9¼ x 6¼ in., 15 + 595 pp. Chicago, Myron C. Clark Publishing Company, 1913. $4.00.

The preface states that the author's intention in this book is to set forth in detail all the principal points on which information is likely to be wanted by an inspector on concrete construction and all the essentials governing the construction of reinforced concrete structures. The subject-matter is said to consist of a series of rules and directions to be followed in inspecting concrete construction, with brief explanations of the reasons for each rule and of their importance, and an endeavor has been made to observe a logical order and a due proportion between the different parts in order to produce the work in a form for convenient use and ready reference. Some of the matter, it is stated, has been collected from engineering periodicals and from the publications of engineering societies, but much of it has been taken from the personal records of the author, and he hopes that the book may be found useful by those engaged in the inspection, supervision, and construction of work involving the use of concrete. The Chapter headings are: Inspection of Hydraulic Cement; Inspection of Sand, Stone and Miscellaneous Concrete Materials; Inspection of Proportioning and Mixing Concrete; Inspection of Forms, Molds, Centering and Falsework; Inspection of Steel Reinforcement; Inspection of Concreting; Inspection of Surface Finishes for Concrete Work; Inspection of Waterproofing for Concrete Work; Inspection of Concrete Sidewalk, Curb, and Pavement Construction; Inspection of Concrete Products; Inspection of Molding and Driving Concrete Piles; Index.

THE ELECTRIC PROPULSION OF SHIPS.

By H. M. Hobart. Cloth, 8¾ x 5½ in., illus., 12 + 167 pp. London and New York, Harper & Brothers, 1911. $2.00.

The attention of engineers is being called, it is stated, with increasing frequency to the question as to whether or not it is commercially advantageous to incorporate electrical apparatus in the machinery used for propelling ships, and in this book, the author has endeavored to show conclusively that such applications are commercially advantageous in many cases, to outline roughly their limitations, and to indicate the boundary line between appropriate and inappropriate cases. The Contents are: Introductory; The Size and Power of Ships; The Energy Required per Ton-Mile in Propelling Ships at Constant Speed; The Frictional Resistance of Ships; The Momentum of Ships; The Speed and Efficiency of Propellers; Mechanical Speed-Reduction Gearing for Steam Turbines; Electrical Speed-Reduction Gearing for Steam Turbines; The Use of Superheated Steam in Marine Engines; Electrical Gear as a Means for Improving the Load Factor; Internal-Combustion Engines for Ship Propulsion; Alternating and Continuous Electricity for Ship Propulsion; Some Systems of Propelling Ships Electrically; The Alter-Phase System for Ship Propulsion; The Durtnall System of Propelling Ships; The Emmet System of Ship Propulsion; Index.

DER DIESELMYTHUS :

Quellenmässige Geschichte der Entstehung des heutigen Oelmotors. Von J. Lüders. Paper, 10 x 6¾ in., illus., 236 pp. Berlin, M. Krayn, 1913. 4.50 Marks.

As stated in the title, this book is a history of the Diesel motor, based on a study of the original and other patents secured on the same by the late Dr. Diesel. The author states in his Introduction that this work is based on scientific and technical proofs which are accessible to every one that, although Dr. Diesel gave the impetus to the perfecting of oil motors and to their practical use, his title as inventor of them needs the correction which has been given in detail in these pages. The Contents are: Einleitung; Verzeichnis der benutzten Schriften; Diesels Patent

* Unless otherwise specified, books in this list have been donated by the publishers.

No. 67 207 vom 28. Februar 1892; Diesels Broschüre (Anfang 1893); Die Kritiker der Broschüre Diesels; Diesels Patent No. 82 168 vom 30. November, 1893; Diesels Versuche mit dem neuen Motor (1893-1897); Schröters und Diesels Vorträge in Cassel (1897); Die Kritiker des Casseler Vortrages; Diesels Vortrag in Berlin (1912); Diesels Schrift: "Die Entstehung des Dieselmotors" (Herbst 1913).

TIN DEPOSITS OF THE WORLD

With a Chapter on Tin Smelting. By Sydney Fawns. Third Edition. Cloth, $8\frac{3}{4}$ x $5\frac{1}{2}$ in., illus., 10 + 306 pp. London, The Mining Journal.

In this, the third edition, the author, it is stated, has made every effort to bring the information contained herein as far as possible up to date, and in such endeavor many of the chapters have been entirely re-written and new and interesting details added. The various methods and machines used in tin mining have been described, it is said, to cover the latest accepted practices. All the statistics have been corrected and the latest authentic published reports from each tin-producing center are included. A bibliography of the subject is also given. The Contents are: The Common Forms of Stanniferous Minerals and the Early History of Tin Mining; Description of Tin Deposits; Alluvial Tin Mining; Alluvial Tin Deposits of the Malay Peninsula; Tin Lode Deposits in the Malay Peninsula; Alluvial Deposits of Banca, Billiton, Singkep, Siak, Sumatra, Siam, British Burma, Shan States, and French Indo-China; Tin Deposits of Bolivia; Tin Deposits of Africa; Tin Deposits of Cornwall; Mount Bischoff Tin Mine; Tin Deposits of Tasmania; Tin Deposits of New South Wales; Tin Deposits of Queensland; Tin Deposits of Western Australia, Northern Territory of South Australia, Victoria, and New Zealand; Tin Deposits of Central Europe, Spain, Portugal, France, Italy, Scotland, Ireland; Tin Deposits of China, Japan, Greenland, Finland, Korea, Siberia; Tin Deposits of Mexico, United States of America, York Regions of Alaska; Tin Crushing and Dressing Machinery; Dredging for Tin; Methods of Tin Assaying; Statistics of Tin Production; Tin Smelting; Bibliography; Index.

THE NEW BUILDING ESTIMATOR:

A Handbook for Architects, Builders, Contractors, Engineers, Superintendents, and Draftsmen. By William Arthur. Eleventh Edition, Revised and Enlarged. Leather, 7 x $4\frac{3}{4}$ in., illus., 705 pp. New York, David Williams Company, 1913. \$3.00.

In a secondary title, it is stated that this book is a practical guide to estimating the cost of labor and materials in building construction from excavation to finish. In addition to dealing with standard construction, detailed figures for reinforced concrete and many chapters on Square, Cubic Foot, Comparative Costs, Physical Valuations, and on Depreciation are given, all of which, it is hoped, will be found useful by architects, contractors, and by railroad engineers. In addition to the ordinary tabulated matter, there are also special tables for quick calculation, including a series for estimating plaster. An hourly wage table is also included, as well as many practical examples of various types of construction. The Contents are: Part I, Approximate Estimating; Part II, Detailed Estimating.

THE MINING WORLD INDEX OF CURRENT LITERATURE,

Vol. III, First Half Year, 1913. By George E. Sisley. Cloth, $9\frac{1}{4}$ x 6 in., 26 + 158 pp. Chicago, Mining World Company, 1913.

This publication, the first volume of which appeared in August, 1912, is, as stated on the title-page, an international bibliography of Mining and the Mining Sciences, compiled and revised semi-annually from the Index of the World's Current Literature, which appears weekly in *Mining and Engineering World*. In this, the third, volume which covers the field of mining, metallurgy, and kindred subjects for the first six months of 1913, but few changes have been made, one new feature being a list of the publications indexed, including periodicals, books, and transactions, bulletins, etc., of schools, societies, and Government bureaus. The subject-matter is divided into classes, under which the entries are arranged alphabetically by subject and author, and includes the title of the article (which, if in foreign languages, is usually followed by translations or explanations in English), a brief amplification or explanation when the title is insufficient, the name of the journal in which the article appeared or where abstracts of it may be found, the date and page number, the approximate number of words in the article, and the price, which arrangement, it is hoped, will be appreciated by those whose library facilities are limited, which condition applies, it is said, at nearly all mines and mining centers. The Contents are: Metals and Metal Ores; Non-Metals; Mines and Mining; Mills and Milling; Chemistry and Assaying; Metallurgy; Power and Machinery; Miscellaneous.

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The Engineering Index Annual for 1913. Compiled from the Engineering Index Published Monthly in the *Engineering Magazine* during 1913. The Engineering Magazine Co., New York, 1914.

The American Year Book: A Record of Events and Progress, 1913. Edited by Francis G. Wickware. D. Appleton and Company, New York and London, 1914.

Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens. Herausgegeben vom Verein deutscher Ingenieure. Heft 145. Julius Springer, Berlin, 1913.

Reports and Orders of the Public Service Commission of the State of New Hampshire for the period ending August 31st, 1913. Vol. 3. New Hampshire Public Service Commission.

Electrical Engineering: The Theory and Characteristics of Electrical Circuits and Machinery. By Clarence V. Christie. McGraw-Hill Book Company, Inc., New York and London, 1913.

Transformer Practice: Manufacture, Assembling, Connections, Operation and Testing. By William T. Taylor. Second Edition, Enlarged and Reset. McGraw-Hill Book Company, Inc., New York and London, 1913.

Mechanical Refrigeration: A Treatise for Technical Students and Engineers. By H. J. Macintire. John Wiley & Sons, Inc., New York; Chapman & Hall, Limited, London, 1914.

A Text-Book on the Method of Least Squares. By Mansfield Merriman, M. Am. Soc. C. E. Eighth Edition, Revised. John Wiley & Sons, Inc., New York; Chapman & Hall, Limited, London, 1913.

Government Ownership of Railways. By Samuel O. Dunn. D. Appleton & Company, New York and London, 1913.

Railway Gazette: A Journal of Transportation Engineering, Docks, Harbours, Contracts and Railway News. Vol. 15-16, July, 1911–June, 1912. London, 1912.

The Sampling and Assay of the Precious Metals; Comprising Gold, Silver, Platinum, and the Platinum Group Metals in Ores, Bullion, and Products. By Ernest A. Smith. J. B. Lippincott Co., Philadelphia; Charles Griffin & Co., Ltd., London, 1913.

Cyclopedia of Civil Engineering: A General Reference Work on Surveying, Railroad Engineering, Structural Engineering, Roofs and Bridges, Masonry and Reinforced Concrete, Highway Construction, Hydraulic Engineering, Irrigation, River and Harbor Improvement, Municipal Engineering, Cost Analysis, Etc. Edited by Frederick E. Turneaure, M. Am. Soc. C. E., Assisted by a Corps of Civil and Consulting Engineers and Technical Experts of the Highest Professional Standing. 8 Vol. American Technical Society, Chicago, 1913.

SUMMARY OF ACCESSIONS

(From February 3d to March 2d, 1914)

Donations (including 20 duplicates)	254
By purchase	20
Total	274

MEMBERSHIP

ADDITIONS

(From February 6th to March 5th, 1914)

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ALEXANDER, HENRY DAVID. Special Res. Engr., New York State Barge Canal, Barge Canal Office, Albany, N. Y.		Feb. 4, 1914
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BAYLEY, EDGAR ALCANDER. Asst. Engr., Dept. of Public Service, City of Los Angeles, 645 South Olive St., Los Angeles, Cal..............................		Dec. 31, 1913
BAYLIS, ARTHUR RAYMOND. Chf. Draftsman, 59th St. Power House, I. R. T. Co. and New York Rys. Co., 600 West 59th St., New York City......................	Assoc. M. M.	June 6, 1906 Feb. 4, 1914
BENHAM, WEBSTER LANCE. Cons. and Superv. Engr.; Chf. Engr., The Benham Eng. Co., 435 Am. National Bank Bldg., Oklahoma, Okla.........................	Assoc. M. M.	April 5, 1910 Feb. 4, 1914
BENNETT, CHARLES JOSEPH. State Highway Commr., State Capitol, Hartford, Conn............................		Feb. 4, 1914
BOOZ, HORACE COREY. Asst. Chf. Engr., P. R. R., Room 613, Broad St. Station, Philadelphia, Pa.....................	Assoc. M. M.	April 4, 1906 Feb. 4, 1914
BOUILLON, ARTHUR MAXIMILLIEN. Dist. Engr., G. T. P. Ry., P. O. Box 55, Quebec, Que., Canada.........................	Assoc. M. M.	June 6, 1911 Feb. 4, 1914
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GEARHART, WALTER SCOTT. State Engr., Kansas State Agri. Coll., Manhattan, Kans.	Assoc. M. M.	June 3, 1908 Feb. 4, 1914
GRAVELLE, ALVIN. Asst. Bridge Engr., City of Cleveland, 411 City Hall, Cleveland, Ohio.................................	Assoc. M. M.	Nov. 6, 1901 Feb. 4, 1914
GUERINGER, LOUIS AMEDEE. Constr. Engr., Frisco Lines; County Engr., Victoria County, Victoria, Tex................	Assoc. M. M.	May 7, 1902 Feb. 4, 1914
HAYDON, GEORGE CONDIT. U. S. Asst. Engr., Missouri River Impvt., 707 Postal Bldg., Kansas City, Mo....		Feb. 4, 1914
MACALLUM, ANDREW FULLERTON. City Engr., Hamilton, Ont., Canada....................................		Feb. 4, 1914
MARANI, VIRGIL GEORGE. Cons. Engr., 702 The 1900 Euclid Bldg., Cleveland, Ohio..........................		Feb. 4, 1914

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NASH, FRANKLYN DANA. Room 408, Merchants National Bank Bldg., Vicksburg, Miss	Assoc. M. M.	May 4, 1909 Feb. 4, 1914
SATTLEY, ROBERT CARLOS. Valuation Engr., Rock Island Lines, Room 1127, La Salle St. Station, Chicago, Ill.		Sept. 3, 1913
SCHUSSLER, HERMANN FREDERICK AUGUST. Civ., Mech. and Hydr. Engr., Room 618, Nevada Bank Bldg., San Francisco, Cal		Feb. 4, 1914
SPIKER, JACOB STEPHEN. Cons. Civ. and San. Engr., Court House, Vincennes, Ind		Feb. 4, 1914
STORRS, JOHN WILLIAMS. Cons. Engr., Concord, N. H.		Feb. 4, 1914

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BAILEY, THOMAS SHERWOOD. Asst. Engr., Dept. of State Engr. and Surv., 2 Cornelius Ave., Schenectady, N. Y	Jun. Assoc. M.	Feb. 28, 1911 Feb. 4, 1914
BARKER, BERTRAND DON. Supt. of Constr., George A. Quinlan, 6200 Ellis Ave., Chicago, Ill		Sept. 3, 1913
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CRAWFORD, WILLIAM HARRISON. Designing and Constr. Engr., Operating Dept., Am. Pipe & Constr. Co., 112 North Broad St., Philadelphia, Pa		Feb. 4, 1914
DEXTER, CHARLES EDWIN. Care, George A. Fuller Co., 38 Taylor Arcade, Cleveland, Ohio		June 4, 1913
DODGE, RALPH EMERSON. Office Engr., California Highway Comm., Forum Bldg., Sacramento, Cal		Nov. 12, 1913
ELLIOTT, ALLEN EDRICK. Asst. Engr., B. & L. E. R. R., Greenville, Pa		Feb. 4, 1914
FIELD, CLESSON HERBERT. Asst. Structural Engr., Lackawanna Steel Co. (Res., 223 Loring Ave.), Buffalo, N. Y	Jun. Assoc. M.	June 1, 1909 Nov. 12, 1913
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KENDALL, FRANK B. Care, Toronto Harbor Commrs., Toronto, Ont., Canada		Feb. 4, 1914
KITTREDGE, RAYMOND BROWN. Asst. Prof. of Railroad Eng., State Univ. of Iowa, 904 Bowery St., Iowa City, Iowa		Feb. 4, 1914
KUHL, HERMAN CHARLES. Care, U. S. Reclamation Service, Gilman, Mont		Sept. 3, 1913
MARTIN, JAMES WALTER. Junior Engr., U. S. Engr. Office, Georgetown, S. C		Feb. 4, 1914
MERIWETHER, BENJAMIN BALDWIN. Engr., Birmingham Realty Co., 2118 First Ave., Birmingham, Ala		Feb. 4, 1914
OLMSTED, HERBERT WARNER. Asst. Engr., Board of Water Supply, City of New York, Pleasantville, N. Y		Dec. 31, 1913
OLDS, ROBERT FRANKLIN. Gen. Supt. of Constr., for Constr. Quartermaster, Fort Mills, Philippine Islands		Nov. 12, 1913
PHILIPS, GEORGE WASHINGTON. Res. Engr., New York Connecting R. R., 462 Amity St., Flushing, N. Y		Feb. 4, 1914
PHILLIPS, JAMES VERNON. Drainage Engr., U. S. Dept. of Agriculture, Savannah, Ga	Jun. Assoc. M.	Feb. 6, 1912 Nov. 12, 1913
RAY, NORMAN GILMAN. Engr. in Chg., Field Parties, Aluminum Co. of America, P. O. Box 297, Massena, N. Y		Oct. 1, 1913
SACHSE, RICHARD. Prin. Asst. Engr., California Railroad Comm., 1526 Grant St., Berkeley, Cal		Nov. 12, 1913
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YOUNG, FREDERICK CHARLES. Instr. in Civ. Eng., Coll. of Applied Science, State Univ. of Iowa, Iowa City, Iowa	Feb. 4, 1914

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FOUGNER, NICOLAY KNUDTZON. Travelling Mgr., The Trussed Concrete Steel Co., Detroit, Mich., Care, Arnhold Karberg & Co., Hongkong, China.

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RESIGNATIONS

ASSOCIATE MEMBERS	Date of Resignation.
ADELHEIM, WILLIAM THOMAS	Feb. 4, 1914
COX, CHARLES BARROWS	Feb. 4, 1914
HOLTSMARK, ERLING	Feb. 4, 1914
SEARS, ROBERT HUMPHRY	Feb. 4, 1914
SOUTHER, THEODORE WHEELER	Feb. 4, 1914
JUNIORS	
BARNES, HARRY EVERETT	Feb. 4, 1914
JONES, WILLIAM HENRY	Feb. 4, 1914

DEATHS

GAZLAY, WEBSTER. Elected Member, June 7th, 1905; died February 17th, 1914.

MELVIN, DAVID NEILSON. Elected Member, July 3d, 1878; died January 27th, 1914.

Total Membership of the Society, March 5th, 1914, 7 335.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(February 2d to March 1st, 1914)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette, Mechanical Edition,* New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.

(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 10c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.

LIST OF ARTICLES

Bridges.

The Fall River Bridge.* James W. Rollins. (109) Feb.
Frankignoul Enlarged-Base Reinforced Concrete Piles.* Frankignoul. (Paper read before the Belgian State Railways.) (21) Feb.
Bay City Bridge, G. T. Ry.* (87) Feb.
Concrete Practice No. 10, Philadelphia & Reading Ry. (Bridges and Culverts).* A. M. Wolf. (87) Feb.
The Crooked River Arch.* C. E. Chase. (19) Feb. 7.
Design and Construction of the Bush and Gunpowder River Bridges Consisting of a Series of Reinforced Concrete Slab Spans.* (86) Feb. 11.
Specifications for Unit Stress in End Posts of Railroad Bridges. Albert Smith. (Paper read before the Indiana Eng. Soc.) (86) Feb. 11.

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*Illustrated.

Bridges—(Continued).

Design Features of the Reinforced Concrete Viaduct on Hopple St., Cincinnati, Ohio.* (86) Serial beginning Feb. 11.
The Rusting of Steel Bridges.* Clifford Older. (Abstract of paper read before the Ill. Soc. of Engrs. and Surveyors.) (13) Feb. 12.
Design and Cost of Ornamental Arch Bridges in Los Angeles.* R. W. Stewart. (14) Feb. 14.
Schuylkill River Railroad Bridge.* (14) Feb. 14.
Standard Concrete Piles for Bridge Foundation, Adopted by Chicago, Burlington & Quincy Railroad. (14) Feb. 14.
Reinforced-Concrete Columns for Bridge over St. Croix River at Hudson, Wisconsin.* (14) Feb. 14.
Bridge over Burrard Inlet at Second Narrows, Vancouver, B. C.; Record-Breaking Swing-Span.* (13) Feb. 19.
Erecting a Long Steel Girder Span for the Duluth, South Shore & Atlantic near Shilo, Wis.* (15) Feb. 20.
Report of Creosoted Piling in Santa Fé Galveston Bay Bridge.* F. B. Ridgway. (Paper read before the Am. Wood Preservers' Assoc.) (15) Feb. 20.
Building a Concrete Bridge in Halves.* (14) Feb. 21.
Separation of Grade Crossings at Chattanooga, Tenn., McCallie Avenue Viaduct.* (18) Feb. 21.
Interchanging Two Bridge Spans, Wabash R. R. Bridge at Hannibal, Mo.* F. W. Curtis. (13) Feb. 26.
Recommendations for Reinforced-Concrete Highway Bridges and Culverts. (Report of Comm. to Am. Concrete Inst.) (13) Feb. 26.
Steel Highway Bridge Failure.* N. B. Garver. (14) Feb. 28.
Le Pont en Arc de Hell Gate sur l'East River à New-York.* P. Calfas. (33) Jan. 31.
Viaducs en Béton Armé du Grundjetobel et de Langwies sur la Ligne de Coire à Arosa (Grisons, Suisse).* A. Bidault des Chaumes. (33) Feb. 7.
Abriss einer allgemeinen Theorie des eingespannten Trägers mit räumlich gewundener Mittellinie.* H. Marcus. (49) Pt. 1.
Der Einfluss der Längs- und Querkräfte auf statisch unbestimmte Bogen- und Rahmentragwerke.* R. Rueb. (79) Vol. 22.
Der Neubau der Draubrücke in Marburg.* Anton Hochsinger. (40) Nov. 22.
Der Betrieb der Brückenwehre im Mohawk-Flusse.* D. A. Watt. (48) Jan. 17.
Provinzialstrassenbrücke Mestre-Mirano (Italien).* Theodor Gesteschi. (78) Jan. 20.
Die neue Strassenbrücke über die Spree in Fürstenwalde.* Karl Bernhard. (51) Serial beginning Feb. 4.
Viktoriabrücke in Bromberg.* Lewe. (80) Feb. 5.

Electrical.

Overhead Wire Construction for Medium and Low Pressures.* A. P. Trotter. (Paper read before the Institution of Post-Office Elec. Engrs.) (26) Jan. 23.
Protective Reactances in Large Power Stations.* James Lyman, Allen M. Rossman and Leslie L. Perry. (42) Feb.
Outdoor *vs.* Indoor Substations. Alexander Macomber. (42) Feb.
Indoor and Outdoor Substations in Pennsylvania.* H. L. Fullerton. (42) Feb.
Outdoor Substations in New England. Fred L. Hunt. (42) Feb.
Outdoor Substations in the Middle West.* Leslie L. Perry. (42) Feb.
Problems of High-Tension Transmission Lines. P. W. Sothman. (42) Feb.
Practical Operation of Suspension Insulators.* H. W. Buck. (42) Feb.
Distribution of Electrical Energy. P. Junkersfeld. (42) Feb.
Effect of Electrical Engineering on Modern Industry. Charles P. Steinmetz. (3) Feb.
The Skin Effect and Alternating-Current Resistance.* Edwin F. Northrup and John R. Carson. (3) Feb.
On a Variable Condenser with a Square Law.* W. Duddell. (77) Feb. 2.
The Theory of the Three-Phase Synchronous Machine.* T. F. Wall. (77) Feb. 2.
Some Phases of the Illumination of Interiors.* Preston S. Millar. (Paper read before the Am. Gas Inst. and Illuminating Eng. Soc.) (24) Feb. 2.
Reactance and Reactance Coils in Power Circuits.* E. P. Hollis. (77) Feb. 2.
Electric Train-Lighting Systems.* T. Ferguson. (77) Feb. 2.
A New Electric Carrier for the Transportation of Mails.* (12) Feb. 6.
Recent Developments in Electricity Supply in Hull.* (26) Serial beginning Feb. 6.
Electric Headlights. J. G. Scrugham. (111) Feb. 7.
Steel Bars or Rails as a Current-Carrying Medium in Mill Haulage.* G. E. Gunning. (62) Feb. 9.
Speed Control of Electric Motors.* (57) Feb. 13.
Testing Reinforced Concrete Poles.* (14) Feb. 14.
Recent Developments in Constant-Current Transmission.* Louis Bell. (27) Feb. 14.
Safety Regulations for Electric Power Companies. W. K. Freudenberger. (111) Feb. 14.
Suggested Methods of Improving the Telephone Service.* W. Aitken. (77) Feb. 16.

*Illustrated.

Electrical—(Continued).

A Cœur D'Alene Electric Plant. Girard B. Rosenblatt. (103) Feb. 21.
Union Central Life Building, Cincinnati (Electrical Features).* (27) Feb. 21.
Resonance in Radiotelegraphy.* J. Hettinger. (27) Serial beginning Feb. 21.
A Regulation Chart for 50-Cycle Lines.* H. B. Dwight. (111) Feb. 21.
Elektrizitätsversorgung der Grossstädte. G. Klingenberg. (41) Serial beginning Jan. 22.
Das Tonrad als Detektor in der drahtlosen Telegraphie.* Rudolf Goldschmidt. (41) Jan. 22.
Ein neuer Drehstrom-Kommutator mit Nebenschlussregulierung durch Bürstenverschiebung.* H. K. Schrage. (41) Jan. 22.
Die Entwicklung des Telegraphenbetriebes auf den grossen unterirdischen Linien Deutschlands.* Otto Arendt. (41) Jan. 22.
Methode zur Ermittlung der Senkregulierwiderstände bei Hubwerken mit elektrischer Senkbremsung.* Ernst Schwarz. (41) Jan. 29.
Rechnerische Ermittlung der sparsamsten Beleuchtung für einen gegebenen Lichtbedarf.* H. v. Glinski. (41) Jan. 29.
Zur Theorie des Saitengalvanometers.* Rudolf Förster. (41) Feb. 5.
Grosse amerikanische Drehstrom-Turbo-Dynamos.* Robert Pohl. (41) Feb. 12.
Ueber Einschaltvorgänge und elektrische Wanderwellen.* Ludwig Binder. (41) Serial beginning Feb. 12.

Marine.

The Manufacture of Armour-Piercing Projectiles.* Leandro Cubillo. (71) Vol. 88.
The Use of Special Steel in Warships.* Hiraga. (Paper read before the Japanese Inst. of Naval Archts.) (11) Feb. 6.
Three-Masted Schooner *Aosta* with Savoia Diesel Marine Engines.* (11) Feb. 6.
The Car Ferry in Freight and Passenger Service.* Harry Chapin Plummer. (19) Feb. 7.
Les Nouveaux Remorqueurs de Rhone de la Cie. Lyonnaise de Navigation et de Remorquage.* A. Dumas. (33) Jan. 31.
Schleuse und Trockendock bei Keokuk, Iowa.* Eger. (40) Jan. 21.

Mechanical.

The Manufacture of Coke in Belgium.* Evence Coppée. (71) Vol. 88.
The Wolseley Motor-Car Works Extensions.* (11) Jan. 23.
Eddies in Gaseous Fluids.* J. G. Clark. (Paper read before the London and Southern District Junior Gas Assoc.) (66) Jan. 27.
Pneumatic Tools.* H. I. Brackenbury. (Paper read before the Coventry Eng. Soc.) (11) Jan. 30.
The Green Aeroplane Engine.* (11) Jan. 30.
The Prudhon Combined Cylindrical and Water-Tube Boiler.* (11) Jan. 30.
The Resistance of Bars, Struts, and Wires in a Current of Air.* A. P. Thurston. (11) Jan. 30.
Modern Gearing.* H. Hubert Thorne. (Abstract of paper read before the Junior Institution of Engrs.) (47) Jan. 30.
Power Production in the United Kingdom.* Edward G. Hiller. (Paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 30; (22) Jan. 30; (57) Feb. 6.
The Present Status of the Wood Turpentine Industry. E. H. French and James R. Withrow. (Paper read before the Am. Inst. of Chemical Engrs.) (105) Feb.
Basic Coke. J. R. Campbell. (Paper read before the Coal Min. Inst. of America.) (45) Feb.
The Briquet for Saving Fuel Waste.* Day Allen Willey. (45) Feb.
Powdered Fuel for Railway Shops.* J. G. Contant. (25) Feb.
Recent Economies in the Sugar Industry.* Alfred L. Webre. (Paper read before the Louisiana Soc. of Engrs.) (1) Feb.
Oxy-Acetylene Cutting in Boiler Work.* C. E. Lester. (94) Feb.
Oxy-Acetylene Welding of Boilers.* Henry Cane. (94) Feb.
Olympic Cement Plant, Bellingham, Washington.* (67) Feb.
Mechanical Refrigeration.* H. J. Macintire. (105) Serial beginning Feb.
Some Phases of the Illumination of Interiors.* Preston S. Millar. (Paper read before the Am. Gas Inst. and Illuminating Eng. Soc.) (24) Feb. 2.
Records of the Year in Smoke Abatement in the Cities of Pittsburgh, Chicago, Philadelphia, Baltimore, Cincinnati, Youngstown, Cleveland, Boston, and Milwaukee.* (62) Feb. 2.
The Economic Aspect of the Repair and Maintenance Section of a Gas-Works. F. R. Parsons. (66) Feb. 3.
Plant of Worcester Salt Co.* Charles H. Bromley. (64) Feb. 3.
Cost of Compressing Air by Electricity.* W. C. Lancaster. (64) Feb. 3.
Softening Boiler Feed Water. E. H. Robie. (64) Feb. 3.
Motor Fuels, with Special Reference to Alcohol. William Reginald Ormandy. (29) Feb. 6.

*Illustrated.

Mechanical—(Continued).

Ball Bearings.* David Martin. (Paper read before the Scientific Soc. of the Royal Technical College.) (47) Feb. 6.

Development of the Natural Gas Industry in California and the Building of California's First Large Natural Gas Line.* W. E. Barrett. (24) Feb. 9.

Large-Capacity Gas-Exhausters.* John Foster. (66) Feb. 10; (22) Feb. 13.

Notes on Electricity in Gas-Works. G. Herbert Haynes. (Paper read before the Manchester Junior Gas Assoc.) (66) Feb. 10.

An Experience in Distribution. William Wilson. (Paper read before the Scottish Junior Gas Assoc.) (66) Feb. 10.

Model Mixed-Flow Steam-Turbine Plant.* Warren O. Rogers. (64) Feb. 10.

Comparative Safe Loads for Ball and Roller Bearings.* A. Riebe. (72) Feb. 12.

Experiments in Pre-Heating Clays. Joseph Keele. (Abstract from *Memoir No. 25*, Canada Geol. Survey.) (96) Feb. 12.

Recent Developments in Driving Indian Textile Mills.* (11) Feb. 13.

Causes of Accidents with Airships. Enrico Forlanini. (Translated from *La Lettura*.) (11) Feb. 13.

Turbine Design. Gerald Stoney, M. Inst. C. E. (Abstract of paper read before the Univ. of Manchester.) (11) Feb. 13.

Recent Developments in Grinding Machines.* Thomas R. Shaw. (47) Feb. 13.

The Noise of Motor Traffic.* (12) Serial beginning Feb. 13.

The Properties and Uses of Oil. L. K. Hirshberg. (82) Feb. 14.

Catastrophic Inability in Aeroplanes.* T. W. Lancaster. (Paper read before the British Assoc.) (19) Feb. 14.

The Moving Picture in the Machine Tool Business.* Chester L. Lucas. (From *Machinery*.) (19) Feb. 14.

Report of Committee on Outdoor Lighting, National Commercial Gas Association.* A. H. Humphrey. (24) Feb. 16.

Making the Foundry Pay Dividends. Stuart Dean. (Paper read before the Pittsburgh Foundrymen's Assoc.) (62) Feb. 16.

The New Jersey Commission 90 Cent Gas Rate Order. (83) Feb. 16.

Proposed Regulations for Heating Value of Gas. (83) Feb. 16.

The Heating of Retort-Settings. David T. Marwick. (Paper read before the Scottish Junior Gas Assoc.) (66) Feb. 17.

The Use of Pulverized Coal as Fuel. W. S. Quigley. (Abstract of paper read before the Am. Foundrymen's Assoc.) (13) Feb. 19.

Pulverized Coal as a Fuel for Boilers. R. C. Carpenter. (Abstract from *Sibley Journal of Engineering*.) (13) Feb. 19.

Methods Used in Making the Daimler-Knight Engine.* I. W. Chubb. (72) Feb. 19.

An Unusual Concrete Pipe Plant at Auburn, Wash.* (15) Feb. 20.

The Wanamaker-Curtiss Transatlantic Biplane.* (46) Feb. 21.

The Experimental Laboratory of the Automobile Club of France.* Jacques Boyer. (19) Feb. 21.

Automatic and Distance Lighters.* Francis H. Gilpin. (Paper read before the Am. Gas Inst.) (24) Feb. 23.

Report of the Committee, Am. Gas Inst., on Power Economies in Gas Plants. (24) Serial beginning Feb. 23.

Economical Burning of Fuel. J. P. Greenwood. (Abstract of paper read before the Practical Refrigerating Engrs. Assoc.) (64) Feb. 24.

Wisconsin Steel Company's New Bar Mill.* (20) Feb. 26.

Développement de l'Aviation.* Rodolphe Soreau. (32) Dec.

Commission de l' Unification Internationale des Pas de Vis des Tuyaux, IVe Session—17 et 18 novembre 1913.* (32) Dec.

Etat Actuel de la Liquéfaction de l'Air et de ses Applications. Georges Claude. (32) Dec.

Note sur la Compression des Gaz Réels. Henri Brot. (37) Dec. 31.

Théorie Mathématique des Machines à Air Liquide et la Détente des Gaz.* Henri Brot. (92) Jan.

Utilisation des Gaz Brûlés des Fours Tournants à la Production de la Vapeur. (84) Jan.

Exposé des Différentes Méthodes de Mesures de l'Energie Consommée par les Fours à Coke à Régénération de Chaleur; Méthode Nouvelle pour la Détermination Exacte de Cette Quantité de Chaleur.* E. Lecocq et J. Pieters. (93) Jan.

Les Récents Travaux du Laboratoire Cryogène de Leyde (Pays-Bas).* C. A. Crommelin. (33) Jan. 24.

Les Machines Frigorifiques à Compression et leurs Perfectionnements Récents.* Henri Brot. (37) Jan. 31.

La Recherche des Erreurs dans les Indicateurs Dynamométriques.* James G. Stewart et O.-H. Wildt. (37) Jan. 31.

Chaudières Multitubulaires, Système J. Van Oosterwyck.* (34) Serial beginning Feb.

Verdampfungsversuche von L. Weiss und das Verdampfungsgesetz für ortfeste Dampfkessel.* O. Köchy. (48) Dec. 27.

Rauchkammerlösche als Brennstoff für Dampfkessel.* O. Nerger. (48) Dec. 27.

Naphthalinbetrieb bei Motorfahrzeugen.* A. Heller. (48) Jan. 3.

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Versuche über den Einfluss der Kompression und der Oberflächen, an denen sich der Wärmeaustausch im Dampfzylinder vollzieht, auf den Arbeitsvorgang einer Einzylinder-Maschine.* E. Heinrich. (48) Serial beginning Jan. 3.
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Die Messung der mechanischen Leistung durch elektrische Pendelmaschinen.* Langer und Finzi. (48) Jan. 10.
Die Instandhaltung der Sägeblätter der Holzbearbeitungsmaschinen.* John Schapiro. (52) Serial beginning Jan. 15.
Lokomotiv-Hebekrane.* Ernst Wülfrath. (48) Jan. 17.
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Untersuchungen an der Trocken-Gasreinigungsanlage auf der Halberger Hütte.* F. Mayer. (50) Feb. 5.
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Metallurgical.

On the Use of Oxygen in Blast-Furnace Practice. Gustave Trasenster. (71) Vol. 88.
A New Method for the Determination of the Critical Points Ar 1 Ac 1.* J. E. Stead. (71) Vol. 88.
On Modern Open-Hearth Steel Furnaces. Benjamin Talbot. (71) Vol. 88.
Heating and Cooling Curves of Manganese Steel. *Sir* Robert A. Hadfield. (71) Vol. 88.
The Distillation of Tar in Metallurgical Practice.* Gevers-Orban. (71) Vol. 88.
The Utilization of Blast-Furnace and Coke-Oven Gases in Metallurgy.* E. Houbaer. (71) Vol. 88.
The Crystallising Properties of Electro-Deposited Iron. J. E. Stead and H. C. H. Carpenter. (71) Vol. 88.
The Allotropic Transformations of Iron.* Albert Sauveur. (71) Vol. 88.
Historical Survey of the Metallurgy of Iron in Belgium. De Laveleye. (71) Vol. 88.
Reinforced Pile Foundations for Blast Furnaces.* Armand Baar. (71) Vol. 88.
Commercial Production of Sound Steel Ingots.* Emil Gathmann. (71) Vol. 88.
The Electric Refining of Steel in an Induction Furnace of Special Type.* Otto Frick. (71) Vol. 88.
The Effect of Heat Treatment on the Colorimetric Test for Carbon in a 0.32 Carbon Steel. Edward D. Campbell and Frank D. Haskins. (71) Vol. 88.
The Manufacture of Armour-Piercing Projectiles.* Leandro Cubillo. (71) Vol. 88.
Application of Clean Blast-Furnace Gas. D. Neustadter. (Paper read before the Cleveland Institution of Engrs.) (22) Jan. 23.
High-Temperature Resistivity of Refractories; a New Method of Measuring with Results for Alundum.* Edwin F. Northrup. (105) Feb.
Dry Chloridization of Ores.* John L. Malm. (105) Feb.; (82) Feb. 14.
Electric Iron Smelting at Hardanger in Norway.* Joh. Härden. (105) Feb.
Concentration of Carnotite by Wet and Dry Methods. (105) Feb.
Some Present-Day Metallurgical Problems. D. A. Lyon. (3) Feb.
The Works of the Alton Steel Company.* (20) Feb. 12.
The Globe Mine and Mill.* Wallace Macgregor. (103) Feb. 14.
Antimony; Its Ores, Metallurgy, and Uses. L. C. Mott. (Paper read before the Am. Chemical Soc.) (103) Feb. 14.
Copper Production of Great Cobar, Ltd.* C. A. Tupper. (82) Feb. 21.
Cyanide Plant of the Cornucopia Mine.* Paul W. Gaebelein. (16) Feb. 28.
Tube-Milling Practice. H. S. Gieser. (16) Feb. 28.

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The Manufacture of Armour-Piercing Projectiles.* Leandro Cubillo. (71) Vol. 88.
La Métallurgie au Point de Vue de l'Artillerie.* L. Regnault. (32) Dec.
Les Types Récents de Canons de Montagne.* D. Duaner. (Abstract from *Revue d'Artillerie.*) (33) Jan. 24.

Mining.

Firedamp in Coal-Mines, and the Prevention of Explosions. John Harger. (Paper read before the Manchester Geol. and Min. Soc.) (106) Vol. 46, Pt. 2.
The Detection of Gob-Fires. John Harger. (Paper read before the Manchester Geol. and Min. Soc.) (106) Vol. 46, Pt. 2.
Notes on a New Process for the Washing of Coal at the St. Nicholas Pit of the Societe des Charbonnages de l'Esperance et Bonne Fortune, near Liege, Belgium. Leo Dorey Ford. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 46, Pt. 2.

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The Effect of Increased Atmospheric Pressure on the Height of the Gas-Cap.* George A. Lodge. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 46, Pt. 2.
An Account of the Use of Rescue-Apparatus at Lodge Mill Colliery, Huddersfield. W. D. Lloyd. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 46, Pt. 2.
New Method of Making Large Arches in Mines.* John Roberts. (Paper read before the South Wales Inst. of Engrs.) (22) Jan. 23; (57) Jan. 30.
Spontaneous Combustion of Coal in Mines: Report of the Departmental Committee of the United Kingdom. (22) Jan. 30.
Unwatering Old Workings at Middleton Colliery.* Cecil Hinchliffe. (Paper read before the National Assoc. of Colliery Managers.) (22) Jan. 30.
Different Types of Skips Used at Butte.* Claude T. Rice. (82) Jan. 31.
Prevention of Dust and Gas Explosions. Geo. S. Rice. (82) Jan. 31.
The Kieselguhr Industry.* Percy A. Boeck. (105) Feb.
French Coal Dust Precautions.* Pol Dunaime. (45) Feb.
Hisylvania Mine No. 23.* Wilbur Greeley Burroughs. (45) Feb.
Mining at Lowe, West Virginia.* William Z. Price. (45) Feb.
The Effects of the Detonation of Gun-Cotton.* B. Hopkinson. (Paper read before the North-East Coast Institution of Engrs. and Shipbuilders.) (11) Feb. 6.
Mining the Iron-Ore Bodies of Cuba.* C. A. Tupper. (82) Feb. 7.
The Design of Gold Dredge Buckets.* George E. Sibbett. (16) Feb. 7.
Status and Plans of the Alaska Juneau Gold Mining Co.* Robert A. Kinzie. (16) Feb. 7.
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Electric Pumping Plant at Tilmanstone Colliery.* H. J. Wroe. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Feb. 13.
Mining and Treatment of Lead and Zinc Ores in the Joplin District, Missouri. Clarence A. Wright. (Abstract of paper from U. S. Bureau of Mines.) (68) Feb. 14.
Messina Copper Mine, Northern Transvaal.* Owen Letcher. (103) Feb. 14.
Accident Prevention at the Nevada Consolidated. Lindsay Duncan. (Paper read before the Industrial Safety Conference.) (103) Feb. 14.
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Mining Methods in the Bering River Coalfield, Alaska.* W. R. Crane. (103) Feb. 21.
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The Limitations of Mathematical Theory Applied to Engineering. W. G. Button. (2) Jan.
The Engineer's Part in the Regulation of Public Utilities. (Papers read before the Technical Conference at Stevens College.) (8) Jan.
The Status and Duty of the Engineer. J. A. Ockerson. (Paper read before the Engrs. Club of St. Louis.) (1) Feb.
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An Organ on Which Color Compositions are Played.* John W. N. Sullivan. (46) Feb. 21.

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Concrete Roadways.* Lewis R. Ferguson. (2) Jan.
Colossal Waste Due to Bad Municipal Engineering. Bernard J. Newman. (2) Jan.
Road Signs.* Arthur E. Collins, M. Inst. C. E. (104) Jan. 30.
The Legal Precedents of 1913 in Relation to Municipal Engineering. J. B. Reignier Conder. (104) Jan. 30.
The Relative Economy of Constructing Bituminous Pavements by Penetration Methods.* George C. Warren. (Paper read before the Am. Assoc. for the Advancement of Science.) (60) Feb.
Pavements of Trenton, N. J.* Harry F. Harris. (60) Feb.
Methods and Cost of Constructing a Sand-Gumbo Road in Nebraska and Methods and Cost of Constructing a Brick-Cinder Road in Mississippi. (Abstract from *Bulletin No. 53,* U. S. Dept. of Agriculture.) (86) Feb. 4.

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Method and Cost of Constructing Concrete Road with Bituminous Wearing Surface in California.* C. L. Rakestraw. (86) Feb. 11.
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Concrete Road Construction: a Symposium from Committee Reports to the National Conference on Concrete Road Building. (86) Feb. 18; (13) Feb. 19, Feb. 26; (14) Feb. 28.
Cost of Constructing a Concrete Pavement Surfaced with Asphaltic Oil and Stone Screenings for Huntington Drive, Alhambra, California.* Earnest F. Ayres. (86) Feb. 18.
A Concrete Road at La Salle, Ill. A. H. Hunter. (Abstract of paper read before the Ill. Soc. of Engrs. and Surveyors.) (13) Feb. 19.
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Investigation of the Causes of Expansion and Contraction of Concrete in Concrete Roads with Reference to the Prevention of Cracks.* R. J. Wig. (Report made to the Bureau of Standards.) (86) Feb. 25.
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Plant, Highway and Laboratory Inspection of Bituminous Materials. Francis P. Smith. (Paper read at Columbia Univ.) (96) Feb. 26.
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Teerung in gepflasterten Strassen. F. Zink. (39) Jan. 20.
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Features of the Kalka-Simla Railway.* Lewis R. Freeman. (23) Jan. 23.
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Practical Results of Railway Electrification in Italy. (12) Serial beginning Jan. 23.
Some Limiting Factors in Electric Railway Engineering.* E. V. Pannell. (26) Jan. 23.
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Coupled 2-8-2 Single-Phase Alternating-Current Locomotive for the Rhaetian Railway (Switzerland).* (23) Jan. 30.
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Maintenance of Equipment Costs Compared with Locomotive Fuel Costs for Fiscal Year 1913. (18) Jan. 31.
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Public Relations. Guy E. Tripp. (Paper read before the Am. Elec. Ry. Assoc.) (17) Jan. 31.
The Relations of the Public Service Companies and the Public. Henry W. Anderson. (Paper read before the Am. Elec. Ry. Assoc.) (17) Jan. 31.
Regulation or Profit-Sharing. Halford Erickson. (Paper read before the Am. Elec. Ry. Assoc.) (17) Jan. 31.
The Electric Installations of the Mittenwald Railway. H. Marchand. (88) Feb.
Watering the Tires and Greasing the Flanges of Locomotive Wheels. P. B. d'A. (From *La Technique Moderne.*) (88) Feb.
German and French Express Trains from the British Point of View. Wernekke. (From *Zeitung* des Vereins deutscher Eisenbahnverwaltungen.) (88) Feb.
Review of Traffic Questions. C. Colson. (From *Revue politique et parlementaire.*) (88) Feb.
1 D 1 Tank Locomotive, Series 1 100, of the Dutch States Railway Company.* (From *De Ingénieur.*) (88) Feb.

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Grain Tight Construction for Single Sheathed Box Cars.* W. J. Tollerton. (25) Feb.
Cast Iron Wheel Records. H. H. Vaughan. (Paper read before the Canadian Ry. Club.) (25) Feb.
New 4-4-0 Express Engines, Great Central Railway.* (21) Feb.
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Drainage of Fills at Orangeville and Eldridge, Baltimore Division, B. & O. R. R.* Earl Stimson. (87) Feb.
Railroading Sixty Years Ago. C. D. Purdon. (Paper read before the Engrs. Club of St. Louis.) (1) Feb.
Electric Train-Lighting Systems.* T. Ferguson. (77) Feb. 2.
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Internal Transverse Cracks and Fissures in Rails.* Robert Job. (15) Feb. 6.
Remarkable Railway Progress in Canada. J. L. Payne. (15) Feb. 6.
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Re-Appraisal of Railway Property in Nebraska. E. C. Hurd. (15) Feb. 6.
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The London and North-Western Railway: Dynamometer Tests.* (12) Feb. 6.
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Brake Efficiency, Chilled Iron *vs.* Steel Wheels.* F. K. Vial. (18) Feb. 7; (15) Feb. 13.
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Decision of the Interstate Commerce Commission in the Industrial Railways Case. (18) Feb. 7.
Booster-Control Test on Metropolitan Railway, Paris.* Charles Jacquin. (27) Feb. 7.
Driving and Lining Point Defiance Tunnel.* (14) Feb. 7.
Railroad Fill of 1 500 000 Cubic Yards on Swampy Ground. (14) Feb. 7.
Railway Survey in Northern Patagonia, Argentina.* D. L. Reaburn. (13) Feb. 12.
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Electrification of Heavy Mountain Grades.* Joseph P. Ripley. (15) Feb. 13.
New York Central Station at Rochester, N. Y.* (15) Feb. 13.
The Coleman Cut-Off, Atchison, Topeka & Santa Fé.* J. B. Skeen. (15) Feb. 13.
Finchley Road and Wembley Park Widening, Metropolitan Railway. (23) Feb. 13.
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Steaming Process for Ties and Timber.* J. H. Waterman. (Abstract from Report made before the Am. Wood Preservers Assoc.) (18) Feb. 14.
An Improved Rail-Joint for Portable Tracks.* (62) Feb. 16.
Some Railway Conditions Governing Electrification.* Roger T. Smith. (77) Feb. 16.
Modern Methods of Economical Railroad Location.* H. H. Edgerton, Jr. (86) Feb. 18.
Electric Railway Systems. C. E. Eveleth. (From *General Electric Review.*) (96) Feb. 19.
Interior Transverse Fissures in Rails.* P. H. Dudley. (20) Feb. 19; (15) Feb. 27.
The Actual Service of the Track Spike and Tie Plate. Walter D. Wood. (15) Feb. 20.
Laying Rail on the Lehigh Valley. (15) Feb. 20.
Atlantic Type Locomotives on the Pennsylvania.* (15) Feb. 20; (25) Feb.
The Pennsylvania Railroad's Safety Campaign. (15) Feb. 20.

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Economic Theory and Railway Rate Regulation. G. C. Hand. (15) Feb. 27.
Simplifying the Tonnage Rating Problem.* R. S. Mounce. (15) Feb. 27.
Grade Separation in McKees Rocks, Pa.* (15) Feb. 27.
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Calcul des Appareils de Changement de Voie.* M. Lefèvre et O. Saint-Amand. (38) Serial beginning Feb.
Note sur un Système de Suspension à Flexibilité Variable.* H.-C. Mestre. (38) Feb.
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Versuchsweise Anordnung des Oberbaues mit Buchenschwellen auf den badischen Stattseisenbahnen.* E. Lang. (40) Jan. 24.
Anleitung zur Bearbeitung von Betriebsdienstanweisungen. Heinrich. (40) Jan. 28.
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Die Seigerung in Schienen.* S. Schukowsky. (102) Serial beginning Feb. 1.
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Electrolysis Mitigation in Springfield, Ohio. E. B. Rosa and Burton McCollum. (Abstract from *Bulletin No. 27,* U. S. Bureau of Standards.) (17) Jan. 31.
Track Standards for New Rapid-Transit Lines in New York.* Ellsworth L. Mills. (14) Feb. 7.
Underpinning New Deep Subway Excavation.* (14) Feb. 7.
Alternating-Current Supply in New York City.* (27) Serial beginning Feb. 7.
Rapid Transit Progress in London.* (17) Feb. 7.
The Evolution of Railway Motor Lubrication.* Alfred Green. (Abstract of paper read before the New England Street Ry. Club.) (17) Feb. 7.
Highway and Street-Railway Tunnels in San Francisco.* L. E. Torrey. (13) Feb. 12.
Extension of the Boston Subway.* (18) Feb. 14.
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A Single-Truck Convertible Car for New York Suburbs.* (17) Feb. 14.
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Vorstudien zur Einführung des selbsttätigen Signalsystems auf der Berliner Hoch- und Untergrundbahn. C. Kemmann. (41) Serial beginning Feb. 5.
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A Brief Discussion of Imhoff Tanks.* Leslie C. Frank. (36) Feb.
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Some Features in the Design of Sewer Systems.* C. F. Fisher. (36) Feb.
The Main Drainage Works Proposed for New York. George A. Soper. (109) Feb.
The Refuse Incinerator at Moose Jaw, Canada. F. Cartlidge. (60) Feb.
Some Suggested Designs for Sewage Treatment Plants for Residences and Small Institutions.* Paul Hansen. (Paper read before the Am. Soc. of Agri. Engrs.) (86) Feb. 4.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XL. MARCH, 1914. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

By F. Lavis, M. Am. Soc. C. E.

To be Presented April 1st, 1914.

Synopsis.

The question of the gauge of railways, though not important in the United States, where it was solved many years ago by the adoption of 4 ft. 8½ in. as a standard, is a live issue in South and Central America, where there are lines of nine different gauges, those most in use being 1 meter, 4 ft. 8½ in., and 5 ft. 6 in. During a recent visit to the Argentine the writer made an investigation of this subject, some of the results of which are embodied in this paper.

The railways of South America have been most largely developed, thus far, by European capital and engineers, but, owing to the changed conditions which have placed, or will soon place, the United States in the position of an importer of food supplies and a seeker for markets for its manufactured products, it seems evident that we must take a much greater and more intimate interest in South and Central American affairs. This question of gauge, therefore, is one which may interest many American engineers in the not far distant future.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The subject is treated in the order and under the several different headings mentioned below, the conclusions being briefly summarized as follows:

Curvature.—Within the limits of the practical operation of railroads, any radius curvature which is feasible on meter gauge is as practical on 4 ft. 8½-in. or even 5 ft. 6-in.

Gradients.—The working of trains on steep gradients is not affected by gauge, except in so far as the narrow-gauge cuts down the locomotive capacity and is, therefore, at a great disadvantage.

Location.—In view of the above, the narrow-gauge (3 ft. 0-in. to 3 ft. 6-in.) does not permit better adaptability of the alignment to the conformation of the ground.

Cost of Construction.—This is so little more for 4 ft. 8½-in. than for 3 ft. 0-in., or 3 ft. 6-in., as to be more than counterbalanced by economy in operation.

Speed.—This is an important feature in railway operation, which is adversely affected by narrow-gauge.

Stability.—The relative stability of narrow-gauge trains in motion is much less than that of trains on medium- or broad-gauge, and, consequently, high speeds are impractical on narrow-gauge, and may be dangerous.

Track Stresses.—Much higher track stresses are produced on the narrow-gauge. There is greater impact, and therefore more difficulty in maintaining track on narrow- than on broad-gauge.

Rolling Stock—Cars.—The greater width in proportion to the gauge of narrow-gauge cars is considered a necessary evil rather than an advantage, and it is shown that there is probably no advantage to be gained by making the broad (5 ft. 6-in.) gauge cars any wider than they are at present. In the matter of passenger cars, the narrow-gauge is at a decided disadvantage both as to cost per unit of accommodation and in dead weight per passenger. In freight cars there is little difference in cost within the limits of size possible on the narrow-gauge, though cars of twice the capacity are available for medium- or broad-gauge. The increase in weight of modern freight trains, in order to decrease the cost of operation has made heavier draft rigging necessary, thus tending to increase the proportion of dead to paying load to a much greater extent on cars of small capacity than on the larger ones, than has been the case heretofore.

Locomotives.—The cost per unit of capacity of locomotives is about the same for each gauge, up to the limit of the narrow-gauge, which limit for all types is about half that of the 4 ft. 8½-in. or 5 ft. 6-in. There is an economy in the use of heavier types of locomotives not available for the narrow-gauge, and the use of cars of large capacity is a necessity for cheap transportation.

Train Resistance.—The train resistance is decreased by the use of larger units.

Capacity.—Figures are given tending to show that the relative traffic capacity of narrow-gauge lines is not more than half that of medium- or broad-gauge.

Heavier Engines and Trains.—There is a general discussion showing the part played by the increase of train loads in reducing costs in the United States.

Cost of Operation.—The ton-mile costs on both Indian and Australian railways, as well as those in the Argentine, are less on the medium- and broad-gauge than on the narrow-gauge, although the train-mile costs are often higher.

The lowest ton-mile costs on any railway in the Argentine are more than twice as high as they are in the United States. The saving in cost of operation, due to the use of larger cars and heavier trains, would be considerable, and would in itself go a long way toward meeting the interest charge on the additional cost of the wider gauge.

Cost of Maintenance.—Maintenance of way probably costs more per traffic unit on narrow- than on medium- or broad-gauge on any but lines of very light traffic.

Maintenance of equipment should be less on medium- or broad-gauge by reason of the larger capacity and, consequently, lesser number of locomotives and vehicles, provided there is business to warrant the use of these larger units.

Need of Efficient Transportation Machine.—The necessity of an efficient transportation machine to develop the country properly is pointed out, as well as the part played by the railways in developing the United States, the country most nearly comparable with Brazil and the Argentine.

There are two appendices: Appendix A gives a short account of the history of the gauge question in the United States, with some notes of its present status in India, Australia, South Africa, etc. Appendix

B contains a general description of the topography, physical characteristics, and business conditions of Southern South America, and the development and present situation of its railways.

It is shown that the development and growth of this area is most nearly comparable to that of the United States, and for its exploitation a transportation machine not less efficient than that of the railways of this country is necessary. The growth of the railways of the Argentine is reviewed in some detail, showing their development within the past few years from a series of little local lines which were not much affected by the gauge to a series of fairly extensive systems, where the distances to be covered and the necessary weights of trains to be handled are showing the deficiencies of the narrow-gauge. This growth, and the increasing distances from the seaboard to new areas being opened up, all point to the need of the most efficient transportation system for its proper development.

It is believed to be shown that the narrow-gauge is an inferior transportation machine; that, as far as new lines are concerned, there is little saving and no economy in their construction, and that this gauge is inadequate for the development of areas as large as those under consideration; that the 5 ft. 6-in. gauge offers little if any advantage over the 4 ft. 8½-in.; that, taken altogether, the latter, which has been adopted as the standard in North America, Europe, Western Asia, and Australia, is best adapted to the development of this region; that new lines should be built to this gauge, and that it is economically sound to make the expenditures necessary to change the existing narrow-gauge lines to this standard.

It is pointed out that the cost of changing the gauge of the existing lines cannot be considered entirely alone, and figures are presented showing the estimated additional cost of the whole system of standard-gauge lines at the end of 20 years. The figures tend to show that, including the cost of changing the existing narrow-gauge lines, and taking into account the additional cost of building the new lines of standard- instead of narrow-gauge, that the final standard-gauge system will have cost about $5 000 per km. more than if the narrow-gauge is perpetuated, and that, under certain assumptions, such as the greater need of stone ballast, the earlier requirements of double track, etc., on the narrow-gauge, this estimate would probably be materially reduced.

Figures are given, which, taking into consideration only those items which are readily estimated in money values, tend to show that the saving in cost of operation would be sufficient to cover the interest charges on the additional cost, and, besides all this, there is the fact that the narrow-gauge is an inferior transportation machine not adapted to the development of the transportation system of a continent.

The question of the gauge of railways was settled in North America many years ago, and, so far as regards this country, or even Europe, any discussion of the matter is purely academic. The development of railway transportation in the rest of the world, however, except in certain limited sections, is as yet in its infancy, the only sections of the vast area lying to the south of the United States where any substantial progress has been made being parts of the Argentine, Southern Brazil, Chile, Peru and Mexico, and these by lines of many different gauges, varying from 2 ft. 6 in. to 5 ft. 6 in.

The writer was asked recently to make a study and report on the gauge question, as affecting the future development of the Argentine, and, as the subject is a vital one in all countries south of the United States —or perhaps more correctly south of Mexico—it is thought that the presentation of those parts of the report which are of general interest may not be untimely. This, perhaps, is especially the case now, when the United States is beginning to be forced to look for foreign markets for its manufactures, with South America as the natural field for its enterprise, and when, consequently, the interest which American engineers are likely to have in this development may be of considerable importance.

The probable future growth of the southern part of South America may be fairly comparable with that of the United States during the past 50 years. The importance of the railways as a factor in this latter has been well stated by M. Colson, an eminent French engineer,* who points out how necessary cheap, efficient railway service is in the development of new countries. It is for this reason that American railway engineers, who have been intimately connected with the railway development of their own country, should be interested in the future of South America, as their own experience should enable them to be among the best judges of the necessities of countries which, like

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* See p. 614.

those in southern South America, are entering on an era of expansion comparable only to that of their own.

Europe, like the United States, long ago adopted the 4 ft. 8½-in. gauge as the standard, but the gauge question is still being actively discussed in India, where the standard is 5 ft. 6 in., with a considerable mileage 1 m. and less. Australia, where the gauges are 5 ft. 3 in., 4 ft. 8½ in., and 3 ft. 6 in., according to recent reports, has only recently decided to unify them and adopt 4 ft. 8½ in. as the standard; and Africa, unfortunately, seems to be definitely committed to what one eminent British engineer recently described as "the miserable 3 ft. 6 in. gauge."* China has adopted 4 ft. 8½ in., the Trans-Siberian Railway is 5 ft. 0 in., the South Manchurian Railway and the Korean railways are 4 ft. 8½ in., and the Japanese railways are mostly 3 ft. 6 in., with some 2 ft. 6 in., though it is stated that these latter are to be changed to 4 ft. 8½ in. as the result of their failure to meet the emergency caused by the Russian war.

It is, perhaps, true that the possible use of railways for war purposes can hardly be considered as an important economic factor in determining the basis on which they shall be designed—at least it is hardly as important in America, either North or South, as it might be in Europe or Asia—still the experience of the Japanese, as well as the English in India, tends to show the inferiority of the narrow gauge for transportation, especially in an emergency. In countries growing as rapidly as those of southern South America, this really means that it will not be long before the narrow-gauge lines will reach the same state of inadequacy to meet the natural development and growth of a new country as they have wherever they have been submitted to the emergency test of war.†

This is already apparent on some of the lines in the Argentine, and, in Brazil, the Paulista Railway Company is now reconstructing its narrow-gauge lines west of Rio Claro to 5 ft. 3 in., as it cannot handle the business offered, although the heaviest narrow-gauge equipment, both locomotives and rolling stock, is being used. Some further notes in regard to the experience of other countries with lines of various gauges are given in Appendix A.

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* "The Railway Gauges of India," by Sir Frederick Robert Upton, *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV. p. 196.

† See, also, p. 580.

Some matters treated in this paper are, perhaps, not strictly technical, or may be considered somewhat elementary; but, as this subject is of interest to laymen as well as to engineers, and as the latter may often be called on to present the arguments, for or against the adoption of any particular gauge, to governments, the public, or investors, it has been thought desirable to include them all. In regard to some phases of the technical aspect of the subject, also, certain matters have been included which have long been accepted as almost axiomatic by engineers in North America, though it is the writer's experience that they are not always so accepted elsewhere.

The lengths of the lines of various gauges in the countries to the south of Mexico (which latter is practically all standard (4 ft. 8½ in.) gauge) are shown in Table 1.* Generally throughout the paper, when speaking of narrow gauge, it will be considered to include the 3 ft., 1 m. and 3 ft. 6 in. The medium or standard gauge is the United States, 4 ft. 8½ in. The broad is 5 ft. 3 in. and 5 ft. 6 in. It may be noted that the standard is 5 ft. 3 in. on the Irish Railways, and 5 ft. 6 in. on the Indian Railways.

TABLE 1.—Length, in Miles, of Railroads of Various Gauges in Countries South of Mexico.

	2 ft 0 in.	2 ft. 6 in.	3 ft. 0 in.	Meter.	3 ft. 6 in.	4 ft. 8½ in.	5 ft. 0 in.	5 ft. 3 in.	5 ft. 6 in.
Guatemala.......	....	....	453	....	....	....	....	....	....
Salvador..........	....	....	100	....	....	....	....	....	....
Honduras.........	....	....	24	....	....	....	....	....	....
Nicaragua.........	....	....	....	....	172	....	....	....	....
Costa Rica........	....	....	....	....	564	....	....	....	....
Panama...........	....	....	....	....	....	....	30	....	....
Colombia.........	....	....	340	63	18	....	....	....	....
Venezuela.........	110	....	24	....	199	....	....	....	....
British Guiana....	....	....	....	....	....	60	....	....	....
Ecuador..........	....	....	....	....	278	....	....	....	....
Peru.............	....	....	262	8	15	994	....	....	....
Bolivia............	....	468	54	....	....	....	....	....	....
Chile.............	....	554	....	2 359	279	517	....	....	497
Argentine.........	....	....	....	6 181	....	1 589	....	....	11 942
Paraguay.........	....	....	....	....	....	232	....	....	....
Uruguay..........	....	....	....	....	....	1 374	....	....	....
Brazil............	....	....	....	11 315	....	....	....	286	....
Totals.........	110	1 022	1 257	19 926	1 525	4 766	30	286	12 439

* Compiled from "Universal Directory of Railway Officials."

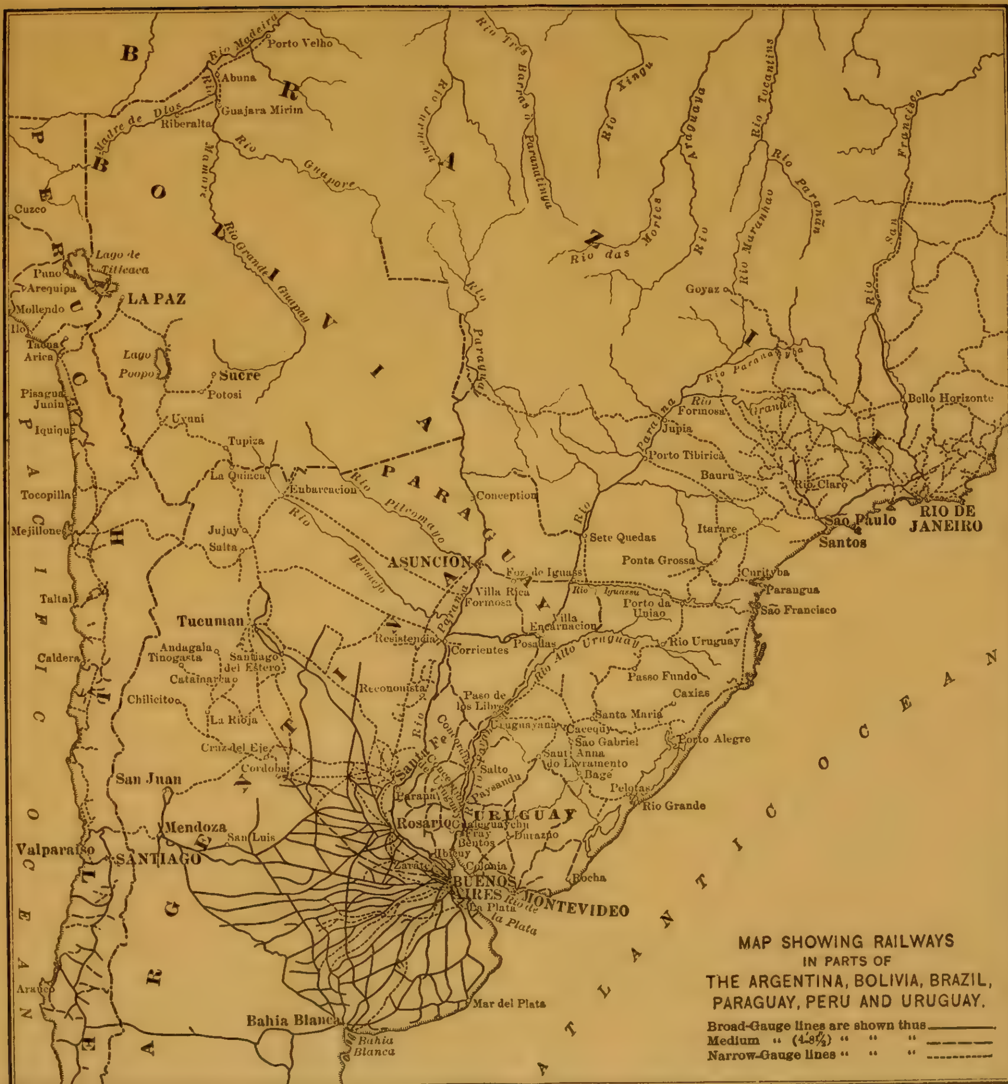
Appendix A contains a table showing the length of the lines of the various gauges throughout the world, the total length of each gauge being, approximately:

Broad	145 300 kilometers	=	90 086 miles.
Medium	727 799 "	=	451 235 "
Narrow	183 416 "	=	113 719 "

The accompanying map, Plate I, shows the railway lines of Southern Brazil, Uruguay, Paraguay, the Argentine, and parts of Chile, Peru, and Bolivia, and indicates the extent of each of the three gauges and their location. Speaking generally, it will be noted that the Brazilian railways are nearly all narrow-gauge, with the exception of parts of the important lines of the São Paulo and Paulista Railways, running into the interior from Santos, the line of the Central Railway of Brazil connecting São Paulo and Rio, and a line from this latter point into the interior, all of these being 5 ft. 3 in. The medium-gauge (4 ft. 8½ in.) lines occupy the strip of country between Brazil and the Parana River, including the whole of Uruguay and Paraguay, and the Argentine Provinces of Entre Rios and Corrientes. In the Argentine, the most intensively developed section—that lying between the City of Bahia Blanca on the south, and the Cities of Rosario, Cordoba, and Mendoza on the north—is occupied principally by the broad-gauge, the narrow-gauge railways occupying the section to the north of this, though in the zone between Buenos Aires, Mendoza, Cordoba, and Santa Fe, the territory is divided between the two, the broad-gauge having somewhat the advantage. Chile and Bolivia seem to be committed to the meter-gauge for all new lines, the former, however, has seven distinct gauges, and in Bolivia the Antofogasta Railway is 2 ft. 6 in. Peru, although fully as mountainous as any part of Chile or Bolivia, has practically adopted the 4 ft. 8½ in. gauge, its lines being among the highest in the world, with several hundred miles at elevations of more than 10 000 ft. above sea level, the famous Oroya Railway reaching 15 583 ft., the highest main line in the world. North of Peru the lines are nearly all narrow-gauge until Mexico is reached, the single exception being the Panama Railway, which is 5 ft.

A more detailed description of the railways of the Argentine, and

PLATE I.
PAPERS, AM. SOC. C. E.
MARCH, 1914.
LAVIS ON
THE GAUGE OF RAILWAYS.



of the climatic, topographical, general physical conditions, and business of the southern part of South America is given in Appendix B.

Effect of Gauge on Location.—It is quite generally supposed that the narrow-gauge is better adapted to mountain location than the wide-gauge, because it permits the use of sharper curves and steeper gradients. Within the limits of practical railroad operation, this supposition is not warranted by the facts, when comparing meter and 4 ft. 8½ in., and there is very little difference with regard to 5 ft. 6 in.

Curvature.—Within the limits of curvature allowed on the meter-gauge roads of the Argentine, that is, with a minimum radius of 490 ft. (150 m.), it is quite possible to operate 5 ft. 6 in. gauge rolling stock which, as a matter of fact, is handled every day in long trains with side-buffers in the yards at the terminals over No. 8 turnouts which have a radius of about 485 ft. (148 m.), and over main-line, No. 10 turnouts with a radius of 780 ft. (241 m.). There are said to be branches on some of the broad-gauge roads with curves of 490 ft. (150 m.) radius; and the line to the power-station at Lules, near Tucuman, over which broad-gauge trains are operated, has curves of less radius than this. The Southern has just located a line with curves of 656 ft. (200 m.) radius across the mountains beyond Neuquen.

The idea still exists in the minds of many people that trains can actually pass sharper curves on a narrow-gauge than on a broad-gauge. "Reduced to absurdity," this, of course, is so, but within the limits of practical railroad operation, on such lines as those under consideration, it is not, and there can be no question that trains of 5 ft. 6 in. gauge with center couplings can pass around any curves which it is practical to operate on meter-gauge lines built to do business. Sharp curvature limits the speed of trains, and in this respect it imposes quite as much, probably greater, limitation on narrow-gauge trains by reason of their lesser stability than it does on broad-gauge trains. The fallacy that sharper curves are not possible on broad-gauge lines probably arose in the early days, before the swiveling truck was introduced. The possible adaptability of the broader or medium gauge to the conformation of the ground has also quite generally been confused by the assumption that, because the gauge was broader the whole road and rolling stock must be of heavy construction; that is, with many people the term "light" railways has

been considered synonymous with narrow-gauge, whereas there is no reason that light rails and light rolling stock cannot be used as well on broad-gauge as on narrow. One has only to consider the rolling stock in use on British Railways 25 years ago, or even to a large extent in England to-day, with the little 5-, 8-. and 10-ton wagons, to realize this. The use of sharp curvature is by no means advocated as desirable, as the objections to its use are fully realized, but the objections to sharp curvature are no less on meter-gauge than on the medium or broad.

Properly designed rolling stock is, of course, a necessity in any case. Certain 40-ton shunting engines of the Cordoba Central Railway (meter-gauge) have difficulty in passing curves of 328 ft. (100 m.) radius in the yards, by reason of their stiff frames and long wheel base. The 850 000-lb. Mallets on the Atchison, Topeka and Santa Fe Railroad in the United States (4 ft. 8½ in.) are operated on curves of only slightly greater radius without trouble, and haul trains of standard Pullman equipment, the coaches being from 65 to 75 ft. long.

A locomotive of the Mikado (2-8-2) type, having a total weight of 285 000 lb. (220 000 lb. on drivers), built by the Baldwin Locomotive Works, has just been put in service on the railway of the Woodward Iron Company (4 ft. 8½ in.), on a line with 3% grades and 16° (110 m. radius) curves.

The New York City Elevated Railways (4 ft. 8½ in.) were built about 35 years ago, and are operated on an open steel viaduct over the streets of New York. Nearly a million passengers and about 1 000 trains a day pass over them. They have several curves of 90 to 125 ft. (30 to 40 m.) radius, and have an enviable record for safety, only one accident to a train in which the life of a passenger was lost having occurred in 35 years. These lines were formerly operated by steam. but now by electricity with multiple-unit trains.

On the lines of the New York City Subway (4 ft. 8½ in.), built about 5 years ago, an average of more than 1 000 000 passengers are carried daily. On these lines the radius of many curves is 147 ft., over which cars 51 ft. long and 9 ft. 0½ in. wide, are operated in 10-car trains by the multiple-unit system.

Table 2 contains some data for standard-gauge lines operating (many with a very heavy traffic) over sharp curves and heavy gradients,

some of them, as will be seen, rising to quite high elevations and thus indicating the adaptability of the 4 ft. 8½-in. gauge to mountain location.

TABLE 2.—DATA RELATING TO GRADIENTS, CURVATURE, ETC., ON CERTAIN RAILROADS.

	Grade.	Radius of Curves. In feet.	Radius of Curves. In meters.	Elevation of summit.	Remarks.
Canadian Pacific	4.49%	498	152	5 299 ft. (1740 m.)	......
" New Line	2.20%	574	175	......	Spiral tunnels.
Atchison, Topeka and Santa Fé	3.30%	357	109	7 510 ft. (2 464 m.)	425-ton Mallet.
Denver and Rio Grande	3.80%	498	150	10 433 ft. (3 423 m.)	......
Colorado Southern	3.50%	193	59	......	Georgetown loop.
Mersey Tunnel	3.00%	......	....	......	......
Mexican Railway	4.75%	......	....	......	......
Nitrate Railway of Chile	4.00%	298	91	......	......
Central Railway of Peru	......	......	....	15 865 ft. (5 205 m.)	......

On the Atchison, Topeka and Santa Fé Railroad—a transcontinental line of very heavy traffic—articulated locomotives of the Mallet type, weighing 425 tons (engine and tender), are in use.

Canadian Pacific: The old 4.49% grade has been operated by adhesion since 1885 up to within a year or two ago, when the spiral tunnels on curves of 574 ft. (175 m.) radius were built and the grade reduced to 2.2 per cent.

On the Nitrate Railway of Chile, 4 ft. 8½-in. gauge, a duplex articulated locomotive hauls 200 tons at 8 miles per hour on 3 to 4% grades (average 2.8%) with curves of 300 ft. radius.

In connection with the location of the line of the Wolgan Valley Railway in Australia, the whole question of gauge and the adaptability of standard (4 ft. 8½ in.) gauge to difficult mountain location was carefully studied by the Chief Engineer, Mr. H. Deane, who reported as follows:

"To bring the line within the region of practicable cost, a ruling grade of 1 in 25 was adopted, with curves of 5 chains [330 ft. = 100 m.] radius. There was no possibility of compensating for curvature, and the 1 in 25 grades occur, therefore, on 5-chain curves, so that the actual ruling grade may be said to be 1 in 22.5 not 1 in 25. A study of plan and section, as well as an inspection on the ground, will show how rigid were the conditions of the problem.

"Bound up with the whole question was that of gauge. Steep grades on a narrow gauge limit the load too much. It was anticipated that when the Company was in full swing, over 1 000 tons of goods would have to be conveyed over the line daily. It was clear, therefore, that the standard gauge must be adopted, especially as the Railway Commissioners had offered to lend their rolling stock if that gauge were not departed from. But how about curvature? it will be said. Was it not excessive? No. Not for the wagons, which were

daily hauled safely over the Camden Railway with its 5-chain curves. But what about locomotives? On the Western line, curves of 8 chains [528 ft. = 173 m.] radius were originally constructed, and had all been cut out because the wear of rails and flanges had been excessive.

"This question had to be solved by looking to the practice of other countries. In New South Wales the locomotives were too stiff. Some other type must be adopted.

"During an extensive journey around the world in 1894, I found numerous curves of 16 degrees, equal to 5½ [363 ft. = 111 m.] chains radius, one curve of 18°, equal to 4.8 chains radius and one of 22°, equal to 4 chains radius, on the Southern Pacific Railway system in the Western United States, and these were traversed by 8-wheeled coupled American locomotives of the Consolidation type. This was rendered possible by providing two of the pairs of wheels with broad, plain treads in place of flanging them. The curves mentioned have now been cut out, but they were worked for many years.

"In 1904 I travelled in a train on the main line of the Canadian Pacific Railway where one curve of 3½ chains [231 ft. = 70 m.] radius exists. All the Company's locomotives traverse this curve.

"On some of the mining branches of the Canadian Pacific Railway, where curves of 5 chains and grades of 4½%, equal to 1 in 22.5, exist, Shay locomotives are used.

"On the Tamaulipas Railway, a scenic line in California, there are curves of 70 and 80 ft. [21 to 25 m.] radius, the traffic being hauled by locomotives of the Shay type.

"On the Kandy Railway in Ceylon there are curves of 5 chains radius, the gauge being 5 ft. 6 in. These are negotiated by locomotives built by Kitson and Company, of Leeds. They are 6-wheeled, coupled, with bogie in front. The middle wheels have thin flanges, considerable play in the axle boxes is allowed, and the connecting rod and side rod pins are barrel shaped, so as to permit of the rods working out of the straight line."

As a further concrete example of the possibilities of operating on standard gauge over mountain lines with heavy gradients and sharp curvature, the following may be cited,* the line being part of one of the Transcontinental routes in the United States:

The main line of the Southern Pacific Railroad in California, between Bakersville and Mojave, a distance of about 68 miles (109 km.), is single track, with maximum grades of 2.2%, uncompensated against traffic both ways, and 10° 20′ (555 ft. = 169 m. radius) curves. There is one continuous curve with a total central angle of 566°, nearly all of which has a radius of less than 600 ft. (180 m.), 18 tunnels

* *Railway Age Gazette*, July 25th. 1913.

with a total length of 8 115 ft. (2 473 m.) and about 4 000 ft. (1 219 m.) of bridging. The summit is 4 025 ft. (1 227 m.) above sea level.

The engines used on this line are as follows:

Passenger service, Mogul (2-6-0) type:

Total weight of engine................	166 320	lb.
" " on drivers...............	144 120	"
Tractive effort.........................	33 320	"

Helper engines, Consolidated (2-8-0) type:

Total weight of engine................	208 000	lb.
" " on drivers...............	187 000	"
Tractive effort.........................	43 305	"

Freight engines, Mallet (2-8-8-2) type:

Total weight of engine................	435 800	lb.
" " on drivers...............	401 000	"
Tractive effort.........................	94 880	"

On a single day in January, 1913, 36 trains passed over this section, of which 16 were passenger trains handling 110 passenger cars, 6 305 tons, and 20 freight trains handling 886 cars, 30 725 tons, a total of 36 trains, 996 cars, 37 030 tons. So that, although sharp curvature is not by any means advocated as desirable, it can be shown that where the exigencies of the topography demand its use, standard-gauge equipment can be operated on it quite as well as on meter-gauge, and the possibilities of reducing operating costs by the use of larger engines far outweigh any small economies in the first cost of narrow-gauge by reason of slightly decreased width of roadbed and shorter ties.

Gradients.—As regards gradients, of course, the gauge makes no difference, the same power being required in each case to move the same weight and kind of train. Wider gauge, however, permits the design of larger locomotives, much beyond the limit of capacity of the narrow-gauge, and the wider firebox and freedom from the cramped space of the narrow-gauge is a decided advantage. It is a fact that standard-gauge locomotives are worked on short stretches, such as approaches to coaling stations, etc., by adhesion, on grades as steep as 10%, and Shay geared locomotives regularly on grades of 4 and 5%; and, as noted previously, there are several lines of heavy traffic

worked by adhesion locomotives on grades between 3 and 4%, and until only recently the Canadian Pacific had grades of 4½% on its main transcontinental line worked by ordinary locomotives. The writer knows of no steeper grade than this regularly worked by adhesion on narrow-gauge.

As in the case of curves, steep gradients are not advocated as by any means desirable, but the ability to operate trains on them is not affected by gauge, except in the one very important item against the narrow-gauge of the very limited capacity of the locomotives as compared with the wider gauge. The heaviest locomotive of the Mallet type, built for narrow-gauge, weighs 176 tons, as compared with 425 tons for the heaviest standard-gauge, or in the proportion of 1 to 2.4.

Speed.—At present there is little demand in the Argentine for speeds in excess of those which can comfortably be maintained on narrow-gauge lines in first-class condition, but the growth of the country is so great that it is rapidly reaching the point where fast speeds will have to be maintained over long distances in order to get passengers and mail to their destinations within a reasonable time. There is no reason to think that the Argentine will long remain content with inferior service, but will expect and will get as good service as is obtained in North America, with which region its transportation system will be most closely comparable.

A tabulation of the regular schedule of the principal long-distance and fast trains in the Argentine at the present time shows the maximum and average speeds to be as follows:

	Kilometers per hour:		Miles per hour:	
	Maximum.	Average.	Maximum.	Average.
Broad gauge..........	61.8	46.3	38.4	28.8
Medium "	30.0	28.4	18.6	17.6
Narrow "	39.6	29.0	24.6	18.0

It is well known that in Europe speeds of 50 to 56 miles per hour are maintained regularly for distances of 200 miles and more by a large number of trains daily. There are many fast trains in the United States, at least 25 or more daily, which do as well or nearly so, though they are much heavier.

On the transcontinental and other long-distance runs in the United States, varying from 1 000 to 2 500 miles, average speeds of 35 to 40

miles per hour are maintained, and daily trains between New York and Chicago on two roads average more than 50 miles per hour for the 1 000-mile run.

Most of the lines in the Argentine, and practically all those cited, have very light gradients and good alignment, whereas, in the United States, most of the lines, especially the transcontinental lines where they cross the Rocky Mountains, have sections of long heavy gradients with considerable curvature; so that, both actually and comparatively, the Argentine is far behind the United States and Europe in the matter of speed, and the narrow-gauge lines are far behind the broad-gauge. The medium-gauge lines of the Entre Rios and the North East Argentine Railways can hardly be considered for comparison at present, as they are not in good physical condition, but they already have in hand plans looking toward an improved service between Buenos Aires and the North which should enable them to run their trains through to Posadas or Corrientes in about 24 hours, or at an average speed of 30 miles per hour, or faster if necessary.

In a discussion before the Institution of Civil Engineers of Great Britain in regard to the gauge of Indian Railways, and in reply to a question as to whether high speed was necessary in that country, the statement was made that, "India, like every other country, had progressive ideas of speed, comfort, and safety, and these would have to be met." There can be no question, also, that this applies equally to the Argentine and Brazil.

For passenger service the narrow-gauge is hopelessly handicapped, and can never meet the requirements of modern railway service either in speed, comfort, or safety. An ordinary through passenger train in the Argentine will consist of about twelve coaches, part of which will be sleepers, diners, etc. Such a train in the United States would weigh between 700 and 800 tons loaded, but, owing to the generally lighter construction of the Argentine rolling stock, the total weight on the broad- or medium-gauge lines there would be about 500 tons, and on the narrow-gauge, with the same type of rolling stock, it would weigh about 400 tons, with less than three-quarters the capacity.

It is possible, on narrow-gauge lines, in first-class condition, with the best design of engine, to reach a maximum speed of 50 miles per hour on a level grade with a comparatively light train (250 to 300 tons).

On grades even as light as 0.5 to 0.6%, which are the ruling ones on most of the lines of heavy traffic of the Argentine, and are the grades on the Central Cordoba Railway line between Buenos Aires and Rosario, the best average speed would hardly be more than 40 miles per hour, which would mean that the Central Cordoba could not, at the best, make the journey in less than 4½ hours, as compared with the present actual time of the Central Argentine Railway of 5 hours, and the possibility of the latter doing it easily in 3½ hours, if necessary, that is, at an average speed of about 50 miles per hour for the run.

With a train of 400 tons, the sustained speed between stations on the narrow-gauge, with very good track, will not exceed 40 miles per hour, and the average speed of express trains, including stops not oftener than, say, once every 62 miles (100 km.) will not be more than 35 miles. This speed might be adequate, but it can only be attained on almost perfect track, and even then at a considerable sacrifice of the margin between safety and recklessness. The effect of every slight irregularity in the track is felt very much more on the narrow-gauge than on medium or broad, and in this respect the broad has an advantage over the medium in a country like the Argentine, where stone or gravel ballast is so expensive.

Stability.—The lesser stability of the narrow-gauge will be immediately realized when one stops to consider that the height of passenger coaches must be practically the same, no matter what the gauge is. Considering the gauge reduced to a minimum with the height remaining the same, it can be seen that a difference in level of the two rails of, say, ¼-in.—less than that of many low joints in earth ballast—would be sufficient to overturn a sufficiently narrow and high vehicle, whereas it would have very little effect on a wide one, even at high speed. Then again, as soon as unevenness develops in the track, the shock caused by passing trains is greater on the narrow-gauge by reason of the high center of gravity, more height in proportion to width, and becomes worse much more quickly, thus increasing the cost of maintenance and making the hard-riding track so often noticed on narrow-gauge roads. Fig. 1 will, perhaps, help to emphasize this point, which is quite important and often lost sight of by those who compare the efficiency of rolling stock on one gauge or the other.

The growing realization of the fact that the speed and weight of modern trains are producing stresses, even in the very best of modern track, far greater than were formerly suspected, is shown by the action of the Canadian Society of Civil Engineers, which, recognizing the probable existence, due to increased loads, of stresses not satisfactorily distributed by the old short ties, has advocated a longer tie.

The Railroad Commissions of the States of Pennsylvania and New York have ordered a reduction in the speed of the fast 18-hour Chicago trains, due to the difficulty of keeping the track in sufficiently good condition to permit these high speeds. The following is from a recent editorial in a technical journal:*

"It seems to us worth while at this time to enter strong protest against any further increase in the loads which are imposed on the steel rails, track and bridges of American Railways."

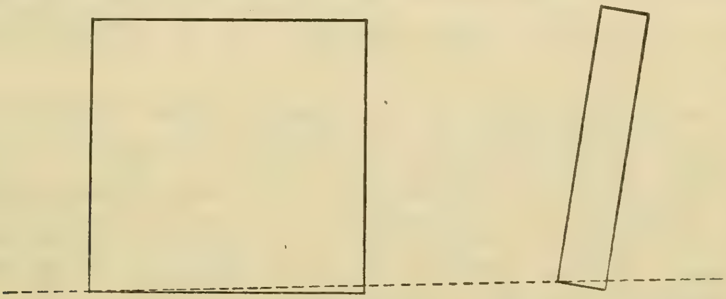


FIG. 1.

In the same journal† there is the following:

"The problem of weight is particularly difficult in regard to passenger locomotives [United States 4 ft. 8½ in.-gauge] * * *. Already we have axle loads of 60 000 to 68 000 lb., and the impact effect or hammer-blow of such loads at high speeds is very severe upon joints, frogs, low spots, and the track in general."

The foregoing comments receive additional force when applied to narrow-gauge roads, where the much greater height in proportion to the width of the vehicles imposes additional shocks and much greater impact stresses than on the wider gauge.

It is well known, of course, that in the transmission of the power from the reciprocating parts of the locomotive engine to the wheels there is an unbalancing of forces which is only partly overcome by the counterbalancing of the wheels. Take, for instance, a Pacific type

* *Engineering News*, December 26th, 1912.

† July 11th, 1912.

engine with 60 000 lb. per axle, running at 60 miles per hour, the rail pressures under each wheel will vary from about 20 000 to 40 000 lb. at each revolution, according to the position of the counterbalance in relation to the rail.

An attempt to overcome this has been made in the class of engines known as balanced compounds, which have been used to a somewhat greater extent in Europe than in the United States. In this type of counterbalanced locomotive the four cylinders are arranged in a row, two inside and two outside the frame, and by the alternating action of the units of each pair they tend to overcome much of the unbalancing of forces noted above. It is hoped that the use of this type will tend to reduce the excessive track stresses with which we now have to contend, but, on account of the necessary arrangement of the cylinders, it cannot very well be adopted for the narrow-gauge, due to lack of requisite width and space for the arrangement of the cylinders.

As a concrete illustration of the maximum speed necessary in order to move goods traffic at even the rather slow average rate of 15 miles (24 km.) per hour, the following extract from a discussion of the problems affecting the transportation of fruit and vegetables in the United States by the Assistant General Manager of the St. Louis and San Francisco Railway, is of interest.*

Citing the run from New Orleans to Denver, 1 515 miles (2 438 km.), he states that at an average speed of 15 miles per hour, which seems quite slow for fruits and vegetables, the total time on this trip would be 101 hours. To get the actual running time there would have to be deducted:

Time lost at 10 district terminals, 1½ hours each	= 15 hours.
Time lost on each of 10 divisions of a single-track road for passing other trains, taking water and coal, say, 2 hours each................	= 20 "
Total	35 hours.

This leaves 66 hours actual running time, or an average speed of about 24 miles (39 km.) per hour. In order to make this average speed, and considering the time lost for slow speeds on heavy grades,

* *Railway Age Gazette*, February 7th, 1913.

slowing down for orders (this particularly in the Argentine with the "Via Libre" system), signals, crossings, junctions, etc., makes it necessary to attain a speed of 40 miles (65 km.) per hour at least, on a good proportion of the distance.

To do that with the average freight train necessitates quite good track—it certainly would be verging on the dangerous to attempt it on even fairly good track on the narrow-gauge.

Rolling Stock.

One of the claims in favor of the narrow-gauge most frequently heard is that of the better utilization of the rolling stock; that the width of the cars is greater in proportion to the width of the gauge than it is on the wider gauges; that there is less dead load to paying load, and often that it is cheaper.

The first of these claims is undoubtedly correct, as far as it goes, but whether or not this is a real advantage is questionable, for, as already pointed out, the impact due to the oscillations caused by imperfections in the track (by reason of the proportionately greater height of the meter-gauge vehicles in relation to the width of gauge) is greater, and it is evident that this is also increased by the greater overhang. This causes a rougher-riding track and increased cost of maintenance, the narrow-gauge being forced to the limit in the matter of overhang, in order to provide reasonable capacity. That the other claims cannot be substantiated is believed to be shown by the following discussion.

Passenger Cars.—When it is considered that for passenger trains the capacity of the narrow-gauge is only three passengers in the same length of coach as will provide accommodation for four on either the medium or broad, it is seen at once that the slight extra cost of construction of the latter is more than justified. Comparing the medium and broad, the medium accommodates four passengers quite comfortably, two on each side of the aisle. the broad-gauge does the same, with some slight excess of room, and it is useless for passenger business to consider increasing the width of the broad-gauge, because, to be effective, this would involve three passengers on one side and two on the other, which, besides making unequal loading, would not be comportable and, therefore, is out of the question. In the dining

cars the medium gauge in the Argentine only allows for three passengers in the width of the car, though four can be accommodated by the broad, and this is the only place where the latter takes any advantage of its increased width. The latest type of all-steel diners in the United States, however, accommodates four passengers, two on each side of the aisle. In sleeping cars, whether of the Pullman type, or with compartments with transverse berths, as in general use in the Argentine, the medium-gauge is quite equal to the broad, and has generally less dead weight per passenger. When divided into compartments, the medium is wide enough to provide comfortable berths crossways of the car, and this is all the broad-gauge can do. If the Pullman type of car is used, the broad-gauge only gives more space in the aisle, which is of no advantage, so that the medium is at a slight advantage over the broad in the sleeping cars. On the narrow-gauge, compartments with berths across the cars are really impractical, although some have been built this way and the space quite ingeniously utilized, yet the berths are not long enough to be comfortable for the ordinary man. The narrow-gauge, therefore, is practically forced to the Pullman type of berths, placed lengthwise of the car, but utilizing only one side of it (as compartments have to be provided) so that they only have about one-half the capacity per unit of length of train that the medium-gauge has, and the dead load per passenger in narrow-gauge sleepers is about 2 tons, as compared with $1\frac{1}{2}$ tons on medium or broad.

In order to determine, as far as possible, the relative capacity, cost, etc., of rolling stock on the different gauges in the Argentine, such information as could be obtained was collected, and is presented herewith for what it is worth. It is not as complete as might be wished, and, of course, comparison of rolling stock in any way is only valuable if the type of construction is the same. Practically, it is obvious, however, that given the same type of construction, the spacing of the wheels can make little if any difference, and these actual figures are given simply as confirmation to show that there is little difference in actual practice. The details of the passenger equipment are given in Table 3.

It will be noted that, in regard to cost, although the difference between the cost per ton as between narrow and medium is about 19%,

the difference in cost per passenger accommodated is 34%, both in favor of the medium-gauge.

Freight Cars.—The advocates of the narrow-gauge, though generally admitting that for passenger business the broader gauges have some advantage, almost invariably claim an equality in transportation of freight, within the limits of their capacity, and that their rolling stock is cheaper and has less dead weight in proportion to carrying capacity.

TABLE 3.—Details of Passenger Equipment.

	Narrow.			Medium.			Broad.		
	Weight, loaded, in tons.	Seats.	Cost, in gold.	Weight, loaded, in tons.	Seats.	Cost, in gold.	Weight, loaded, in tons.	Seats.	Cost, in gold.
First-Class	28	47	$12 250	38.3	68	$12 475	33.8	56	$11 800
Second-Class	27	67	9 900	40.5	102	9 700	32.2	64	8 590
Diners	61	48	29 000	37.7	40	17 000	33.6	32	13 050
Sleepers	27	12	11 760	37.0	20	15 520	32.2	23	12 770

Comparison of Units.

	Narrow	Medium	Broad
Cost per ton......	$440	$356	$350
Cost per passenger..............	361	238	264
Weight per passenger, in tons..	0.82	0.67	0.75

It is generally claimed that in the Argentine there is not much demand at present for either very long trains or cars of large capacity. Leaving this phase of the question for the present, it will be of interest to compare the freight cars of the three different gauges on somewhat the same basis as the passenger cars.

In regard to certain types, some additional data have been furnished to the writer by the Middletown Car Company, which has supplied a large number of narrow-gauge cars to the Government lines, and also to a standard-gauge line in Uruguay and to the broad-gauge port railway at Buenos Aires, these cars being of the same general types of construction. It is to be noted that the comparison of freight cars is somewhat difficult by reason of the greater variation in types of construction, the proportion of tare to total capacity

varying from 40% for well-designed cars to as much as 60% in some of the older types of less capacity. In the United States, also, the cars of large capacity show up to much better advantage than those of smaller capacity, as the underframes and draft rigging have to be the same for all, in order to allow all types of cars to be used in heavy trains. As far as possible, however, from the data of a large number of cars examined, the following are selected as representing good average practice in the Argentine for well-designed cars of fairly large capacity.

The question of what the result would be if the broad-gauge cars were changed so as to get the same proportionate overhang as medium-gauge, is discussed later, and the following comparisons are made on the basis of existing conditions, which it is unlikely will be changed.

It is to be noted that the greatest variation is to be found in the cost of the cars of the Rosario Puerto Belgrano Railway, which is considerably above the average. It is believed that, if sufficient information could be obtained as to the cost of freight cars on the other principal broad-gauge roads of the country, it would show little variation from the costs given of narrow and medium-gauge. The tons are metric, of 1 000 kg. (2 205 lb.).

TABLE 4.—DATA RELATIVE TO COVERED WAGONS (BOX CARS).

	Capacity, in tons.	Area of cross-section, in square feet.	Cubic feet.	Tare, in kilogrammes.	Cost.
NARROW.					
Middletown Car Company......	30	51	1 695	12 200	....
Central Cordoba...............	25	45	1 340	10 500	$1 385
Central Northern...............	20	....	....	10 000	1 100
MEDIUM.					
Middletown Car Company......	30	70.7	2 545	15 380	....
Entre Rios......................	30	73.4	2 350	12 500	$1 470
BROAD.					
Middletown Car Company......	30	71.1	1 860	12 675	$1 400
Rosario Puerto Belgrano.......	40	75.1	2 885	....	2 075
Southern........................	40	....	2 220	15 120	1 875
Central Argentine..............	40	....	2 185	16 350	1 875

It will be seen at once that, as far as regards weight of car to capacity in tons, there is little difference, with the exception of the Middletown Car Company's medium-gauge car, which has very large cubic capacity, the 30-ton cars averaging about 40 to 42 per cent. These small differences are less than are to be found between cars on any one line, of different types of construction.

It will be noted, however, that the narrow-gauge cars have generally only about two-thirds the cubic capacity of the wider gauges. The Middletown Car Company's broad-gauge car is quite a short one, being only 26 ft. long (the usual length being 36 ft., more or less), and, as a large proportion of the goods hauled in box cars is bulky rather than heavy, the extra cubic capacity is a most decided advantage. In the Argentine, as nearly as can be calculated from the statistics published by the Government, more than one-half the freight generally handled in box cars is of the former class, that is, classified by bulk rather than weight.

TABLE 5.—Data Relating to Stock or Cattle Cars.

	Capacity, in tons.	No. of animals.	Width, inside.	Floor area, in square feet.	Tare, in kilogrammes.	Cost.
Narrow.						
Middletown Car Company.........	30	....	7 ft. 11 in.	231	11 984	....
Central Northern..................	20	16	....	190	10 768	$1 240
Central Cordoba....................	22	20	8 ft. 0 in.	232	12 000	1 365
Medium.						
Middletown Car Company.........	26	....	8 ft. 5 in.	295	15 380	....
Entre Rios..........................	22	20	7 ft. 10 in.	256	12 000	$1 475
Broad.						
Rosario Puerto Belgrano.........	40	24	7 ft. 3 in.	281	....	....
Pacific..............................	28	....	9 ft. 6 in.	323	16 000	....
Pacific..............................	28	....	9 ft. 6 in.	314	17 525	....
Average of several................	40	....	8 ft. 6 in.	295	16 250	....

Stock cars are comparable on the basis of the relation between the floor area and the tare weight. The data in Table 5 are all the specific examples the writer has been able to get, and, in spite of being taken

from various sources, they show considerable agreement, the relation between floor area and tare weight for the three types being as follows:

Narrow	51.9 kg. per sq. ft.	(114 lb.)
Medium	49.7 " " " "	(109 ")
Broad	46.9 " " " "	(103 ")

It may be noted that the Entre Rios car is very small for a stock car for 4 ft. 8½-in. gauge, and is not at all representative of the best practice, which would be at least equal to the others. The prices per ton of car and per square foot of floor area work out as follows:

Narrow:	Central Northern	$115.15 per ton	$6.75 per sq. ft.
"	Central Cordoba	113.75 " "	6.40 " " "
Medium:	Entre Rios	118.75 " "	5.75 " " "
Broad:	Rosario Puerto Belgrano	....	....

TABLE 6.—DATA RELATING TO GONDOLAS.

	Capacity, in tons.	Floor area, in square feet.	Cubic feet.	Tare, in kilogrammes.	Cost.	
NARROW.						
Middletown Car Company	30	250	804	10 900	....	....
Central Cordoba	25	244	....	11 000	$1 000	....
Middletown Car Company	30	250	....	12 210	1 345	All steel.
Middletown Car Company	20	....	....	9 070	840	....
MEDIUM.						
Middletown Car Company	30	292	....	12 845	....	....
Entre Rios	30	284	....	11 250	$1 250	....
Rosario Puerto Belgrano	37	308	700	14 400	....	....
BROAD.						
Middletown Car Company	45	....	....	13 800	....	....
Central Argentine	42	336	788	15 240	....	....
Western	45	340	1 630	15 200	....	....

For gondolas the comparison is best made between floor areas and tare weights. The cubic capacity is affected by the height of the sides, and as these may vary quite a little, this is not made, as all the details

are not available, also the loads in these cars are often heaped up above the sides.

Narrow .... 45.8 kg. per sq. ft. of floor area (101 lb.)
Medium .... 43.5 " " " " " " " (96 ")
Broad .... 47.5 " " " " " " " (105 ")

TABLE 7.—DATA RELATING TO FLAT CARS.

	Capacity, in tons.	Floor area, in square feet.	Tare, in kilogrammes.	Cost.	
NARROW.					
Middletown Car Company......	30	262	9 300	....	....
Central Cordoba................	25	256	8 500	$825	....
Central Northern	20	217	8 145	....	....
Chile............................	25	257	8 161	....	All steel.
MEDIUM.					
Middletown Car Company......	30	309	11 570	....	....
Entre Rios.......................	30	258	11 000	$1 150	....
BROAD.					
Rosario Puerto Belgrano.......	40	359	....	$1 915	....
Average of seven cars..........	41	328	13 706	....	....

Narrow .... 34.0 kg. per sq. ft. of floor area (75 lb.)
Medium .... 40.0 " " " " " " " (88 ")
Broad .... 41.7 " " " " " " " (92 ")

It will be noted that, in comparing these four types of freight cars, so far as these figures show, there is very little difference between those of either gauge, what difference there is, however, being in favor of the wider gauges, especially for the box and cattle cars. The data are not as complete as might be wished, but are all that are available of Argentine rolling stock at this time, and, in view of the fact that the information has been taken just as it came, it is believed that it confirms what must be seen to be theoretically correct, that is, that gauge can make very little difference in cars of small capacity. A narrow-gauge box car with the wheels spread out can be used perfectly well on broad-gauge tracks which only differ from its own in being slightly wider. This extra width of trucks, as between narrow and

medium, only involves about 200 to 250 lb. more iron and steel, which at 6 cents per lb. is from $12 to $15, and this extra width is more than compensated by the increased stability and, consequently, lesser impact on the track.

The claim, therefore, that narrow-gauge cars are either cheaper or lighter in weight than cars of the same capacity and same type of construction for wider gauges, does not appear to be sustained by the facts, such variations as are shown being less than those between many cars of different make and design on the same railroad. It is of little value to include for comparison data relating to ordinary United States rolling stock, as this is all generally of heavy type, all the underframes and draft rigging being much heavier than those in use elsewhere, in order that they may withstand the strains due to the use of the cars in heavy trains, and, in addition, all cars are equipped with automatic power brakes, which are not in general use in the Argentine. The railways of the latter country, however, are beginning to realize the values of the saving due to the use of heavier cars and trains, and perhaps more so on the narrow-gauge lines. They are handicapped, however, by the fact that the underframes and draft rigging of all their old rolling stock, and even much of the new, are too weak to stand the strain. They are just beginning to be forced into the same position as the railroads of the United States were some time ago, that is, to adopt the same heavy type of underframe and draft rigging, no matter what the size of the car is, in order to be able to transmit the tractive forces without danger of breaking the train. When this becomes general practice, there will be a much greater discrepancy in the relations between the gross and paying loads for the lighter cars, and they will show up at a much greater disadvantage than they do even now.

Cost of Locomotives.—It is practically impossible to determine with mathematical accuracy the relative economy of locomotives for one gauge or another, as they vary so much in detail. Even supposing three machines as exactly alike as possible, except for the difference in the width apart of the wheels, which would give a difference in cost of only a trifling extra quantity of steel for the wider spacing, the performance would be so different as to vitiate any comparison. Even supposing three machines designed to be as nearly alike as possible

but adapted to the gauge, there would still be so many differences that comparison would be questioned.

It seems to be a perfectly safe assertion, however, that, within the limits of the narrow-gauge, any reputable locomotive builder will undertake to build locomotives of each gauge having about the same power, capacity, and relative fuel economy for about the same price, irrespective of the gauge. There may be a little less steel in the narrow-gauge, but the labor for this slight difference is little, if any, less, and the greater ease of design for the broad-gauge more than compensates for this. There seems to be a vague idea in the minds of some advocates of the narrow-gauge that, by some means or other, narrow-gauge locomotives are more economical, whereas it is probable that the contrary is the case, as is noted in detail later. The hauling capacity of a locomotive, provided of course it is properly designed for power, is determined by the weight on the driving wheels, and a locomotive with 10 tons weight on its driving wheels will have exactly the same adhesion on a wide-gauge that it has on a narrow-gauge, so that, to pull the same gross load, the engines must have the same weight, if they are built on the same plan. A pound of coal, too, will not generate any more steam on the narrow-gauge than on the wide.

In order to get some idea at least of the comparative cost of locomotives for the three gauges in the Argentine, the writer obtained the figures given in Table 8, which, considering the different types of engines involved, different makers, etc., show a sufficient agreement in cost per ton at least not to disprove the argument that gauge makes little, if any, difference. The weights are per ton of engine and tender, light.

Size of Locomotives.—As at present worked, the 5 ft. 6-in. gauge has about the same capacity as the medium, though much heavier engines and trains are operated in the United States on the medium-gauge than elsewhere on any gauge. For reasons to be given later, it is believed that the capacity of the broad-gauge, measured by the size of engines and length of trains, will not be increased beyond that of the medium, unless by some entire change in transportation methods not even guessed at now.

The comparisons in Table 9, therefore, between the largest engines built to date (January, 1913), of the three types most generally in

use, will give a good idea of the tremendous advantage of the wider gauge in this respect for lines of heavy traffic.

TABLE 8.—DATA RELATIVE TO LOCOMOTIVES.

Description.	Class.	Cost.	Weight, in tons.	Cost, per ton.
METER GAUGE.				
Kitson, 1911	4-6-0	$12 500	50	$250
North British, Passenger, 1907	4-4-2	18 750	75	250
" " Cargo, 1907	4-6-2	19 500	78	250
Tank	0-6-4	11 500	47	245
Buenos Aires, Extension Passenger, 1906	4-6-2	18 000	78	230
" " " Cargo, 1906	4-8-0	17 250	73	235
Building Pacific Superheater	4-6-2	19 700	67	295
MEDIUM GAUGE.				
North British, Goods	2-8-0	$19 000	68	$280
" " Mixed Passenger	4-6-0	16 000	65	245
North American, Goods	2-8-0	17 000	56	300
" " Mixed	4-6-0	11 900	56	210
Shunting Engines	0-6-0	8 800	26	335
BROAD GAUGE.				
Tank	2-6-0	$9 360	39.5	$237
"	0-4-0	15 550	63.2	282
Passenger	4-4-0	21 400	70.1	302
Cargo	2-8-0	30 310	91.5	331

In the comparisons in Table 9 the 3 ft. 6-in. engines are given for the Pacific and Mallet types as they are the largest narrow-gauge engines, the largest meter-gauge engines of these types thus far built, however, are quite a little smaller, the meter-gauge Pacific type being 19 by 26 in., with a total weight of engine and tender of 229 500 lb. and the Mallet 18½ by 29 by 22 in., with a total weight of 323 500 lb.

The freight engines of both types on the medium-gauge have practically double the capacity, and though the superiority of the passenger engine in tractive power is not so great, the grate area is just double, 70 sq. ft. as compared with 35 sq. ft., and the total heating surface, 3 936 sq. ft. as compared with 1 981 sq. ft., so that the capacity for sustained work, so necessary in passenger service, is very much greater in proportion. Size of boilers, heating surface, and grate areas are the limiting factors, of course, in all types of locomotives, and though the limit seems to have been nearly reached in these items in locomotives for narrow-gauge, it does not seem to be in those for medium-gauge. It

TABLE 9.—COMPARISON OF LOCOMOTIVES.

Type of Locomotives.	Gauge.	Weight of engine and tender, in pounds.	Weight per driving axle, in pounds.	Diameter of driving wheels, in inches.	Cylinders, in inches.	Maximum tractive power, in pounds.	Maximum load, 0.6% grade, in tons.	Remarks.
Passenger, Pacific, 4-6-2 type	4ft. 8½ in.	469 190	57 440	74	24 by 32	42 000	2 380	Standard-gauge: Baltimore & Ohio R. R. *Engineering News*, July 11th, 1912, p. 49.
	Meter.	259 800	35 330	62	21 by 28	28 800	1 564	3 ft. 6-in. gauge, South African Railways, Am. Loc. Co. Catalogue, p. 35.
Freight or mixed, Mikado, 2-8-2 type for fast or heavy service	4ft. 8½ in.	539 480	53 300	63	28 by 32	27 460	3 196	Standard gauge: Erie R. R., 1912. *Engineering News*. July 11th, 1912, p. 51.
	Meter.	246 600	27 625	49	20 by 26	28 900	1 564	Meter-gauge: Am. Loc. Co. Catalogue, p. 23.
Freight, articulated (Mallet), for heavy, slow service	4ft. 8½ in.	850 000	55 000	57	28 by 36 by 32	111 600	6 324	Standard-gauge: Atchison, Topeka & Santa Fe R. R. *Engineering News*, May 4th, 1911, p. 548.
	Meter.	352 000	32 000	46	18 by 25½ by 26	57 700	3 266	3 ft. 6-in. gauge: South African Railways. Am. Loc. Co. Catalogue, p. 77.

is stated that there is under construction now a medium-gauge locomotive of 160 000 lb. tractive power, nearly 50% more than that of the large Mallet given in Table 9.

Relative Capacity of Wide- and Narrow-Gauge Lines.—The comparisons between the capacities of the meter and broad-gauge lines of India given in Table 10 were made by Sir Robert Upcott,* the figures covering sixteen of the principal railway lines.

TABLE 10.—Carrying Capacities, Compiled from Railway Administration Report, Covering Sixteen Lines in India: Eight of 5 ft. 6-in. Gauge and Eight of Meter-Gauge.

		Ratio. Standard-gauge to meter-gauge.		
Vehicle capacity only	Passenger	1.5	to	1
	Goods.	1.6	to	1
Same, taking speed into account	Passenger	1.9	to	1
	Goods.	1.9	to	1
Gross weight of trains	Passenger	1.4	to	1
	Goods.	2.0	to	1
Same, taking speed into account	Passenger	1.9	to	1
	Goods.	2.4	to	1
Vehicle mileage, loaded and empty	Passenger	1.9	to	1
	Goods.	2.8	to	1
Same, taking speed into account	Passenger	2.4	to	1
	Goods.	3.3	to	1
On actual number of passengers carried 1 mile		2.0	to	1
On actual number of tons carried 1 mile		4.7	to	1

Engines on broad-gauge, average 7 838 000 ton-miles per annum.
Engines on narrow-gauge, average 3 962 000 ton-miles per annum.

The following extract gives the opinion of an important official of the inferiority of the narrow-gauge lines in India, where the problem is very much the same as it is in the Argentine, namely, the movement of very large quantities of grain in a short space of time.

"It has been clearly brought out, writes Major-General Kennedy, by the facts of the present season's famine traffic, that occasions such as the present may arise when, in order to feed a large population, grain has to be carried over single lines of railway for long distances, in quantity sufficient to strain to the utmost the resources of the best appointed lines of the standard 5½-ft. gauge and, as it may be assumed that narrow gauge lines would be far less efficient, the unsuitability of the metre gauge for long lines of through communication cannot be put out of sight when thought is taken for the proper and efficient administration of a country liable to be affected with drought and famine.

* "The Railway Gauges of India," *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV, p. 202.

"Although the railway lines concerned in the present case, *i. e.,* the East Indian, the Great Indian Peninsula and the Madras Railways, have failed to do all that was required of them, or that was needful to meet the full necessities of the case, they have, nevertheless, it may be truly said, largely contributed to save Southern India from a very great calamity, but they have not done this with any margin to spare, or without some injury to particular sections of general trade or to individual traders. The standard gauge lines have been barely able to satisfy essential and urgent demands, and under similar circumstances narrow gauge lines would probably have broken down altogether. Indeed, it is known that in the Southern part of the Madras Presidency considerable inconvenience arose at a break of gauge.

"We may, with the memorial of the Bombay Chamber of Commerce before us, take it as proved that narrow gauge lines are unable to meet the sudden demands likely to be made upon them in India, and that they are far more expensive to work than broad gauge lines. At first sight it is difficult to see how the latter statement can be true, but we think we can clear up the point in a moment. When a given quantity of goods, say grain, has to be conveyed over a railway, two systems of conveyance may be adopted, that is to say, we may run large trains at long intervals or small trains more frequently. Now, on the broad gauge lines in India a goods train complete weighs on the average 505 tons, but on the Rajpootana State metre gauge railway the average goods train weighs but 200 tons, so that the broad gauge trains are really 2½ times as heavy as the narrow gauge trains, while their speed is higher."

Economy of Heavy Engines.—The following statement, recently made by J. J. Hill, F. Am. Soc. C. E.,* one of the foremost railroad executives in the United States, shows the value and importance of heavy engines and wagons of large capacity for the development of cheap and efficient transportation:

"Heavier rails, larger engines, cars of greater capacity, increased train movement and the full utilization of equipment have kept business moving. The density of traffic in England, France and Germany should be as much greater than in the United States as the density in the middle exceeds that in the far western states. Yet here are the facts:

"TON MILES PER MILE OF ROAD.

France	496 939
United Kingdom	529 622
Germany	827 400
United States (1910)	1 071 096

* *Railway Age Gazette*, December 20th, 1912.

"* * * Our railroads move 272 ton miles of freight per dollar of net revenue, where the United Kingdom shows only 58, Germany 172 and France 88.

* * * * * * * * *

"Transportation costs the public from one-third to one-half as much here as in Europe. This cheapness is not purchased at the cost of the workingman. In 1910 the average daily earnings of railway employees in the United States were more than twice as great as in the United Kingdom, and two and three-quarter times as much as on the Prussian-Hesse system in Germany."

There has been some criticism of the exact figures used by Mr. Hill, and of the use of certain units as indices of efficiency, but, making every allowance for this and the fact that such criticisms of details are always possible, in view of the different conditions and manner of computing and compiling statistics, there is not enough difference to affect the main argument, which is, that the use of heavy rolling stock has been the principal cause of the very much lower costs of transportation in the United States than elsewhere.

Fig. 2,* relating to the improvement in operating efficiency of the Chicago, Burlington and Quincy Railroad during the past 10 years, shows with great clearness the possibilities of reduction in operating expenses by the increase of train loads. During this period, as shown by the diagram, the business of the road was nearly doubled (98.3% increase), though there was at the same time an actual decrease of 9.8% in the number of trains required to handle it. Part of this good showing was made by increasing the car loading, that is, the revenue-tons per car-mile, by 45.5% and the remainder was due to increase in the capacity of the locomotives, reduction of grades, and the maintenance of the roadbed and equipment in a high state of efficiency. The average capacity of cars was increased from 23 to 38 tons. The heaviest type of locomotive in use in 1901 developed about 20 000 lb. tractive power; the Mikado (2-8-2) type now in general use has a tractive power of 60 000 lb. The average train load is now 438 revenue-tons per train-mile, as compared with 200 tons in 1901.

Although Canada is somewhat behind the United States in the matter of economical handling of freight, it is far ahead of any other country in this respect. In New South Wales, where they are vig-

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* *Railway Age Gazette*, January 18th, 1913.

orously agitating the double-tracking of many of their lines, the traffic density is only 226 906 ton-miles per mile as compared with 731 776 for Canada, the average train loads being 90 tons and 325 tons, respectively, so that actually more trains are being run in New South Wales to handle a smaller volume of traffic than in Canada, the freight-train-miles per mile of line in New South Wales being 2 512 as compared with 2 252 in Canada, in spite of the fact that the density on the latter is 220% more than on the former, and the average freight charge in Canada is 0.75 cent as compared with 1.78 cents in New South Wales.

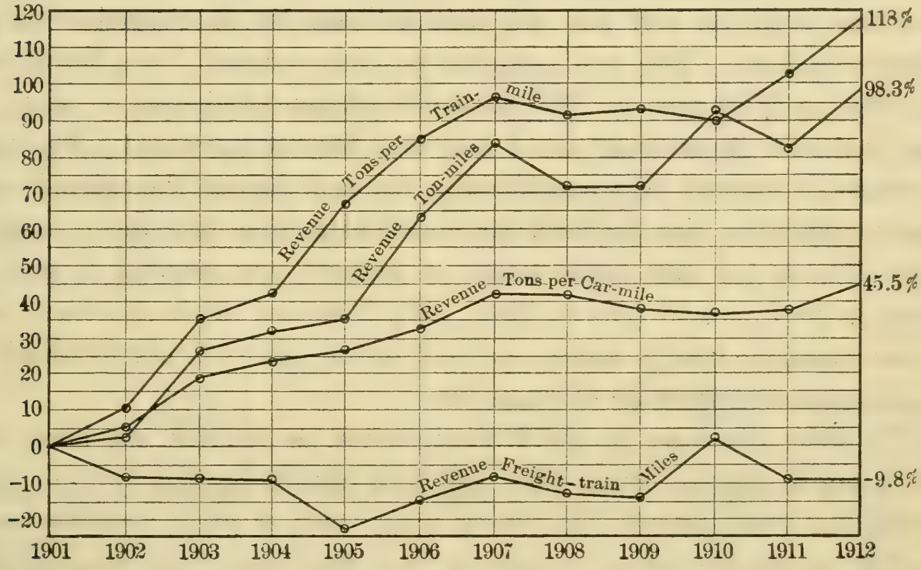

FIG. 2.

It is not only in the United States and Canada, however, that the introduction of heavier locomotives, and consequent increase in train loads, has been found profitable and economical. About 10 years ago, as the result of investigation into the so-called "American" methods of operation, Sir George Gibb, then Manager of the North Eastern Railway of England, introduced changes on that line which have resulted in the increase of the average freight train load from 60 to 95 tons, and on mineral trains from 114 to 184 tons. The result of this* is seen in the fact that for the last half of 1912 a dividend was announced at the rate of 7½% per annum, more than twice the average for British railways.

* *Railway Age Gazette*, March 28th, 1913.

In Germany, also, according to a statement of Mr. Hammer,* there is a growing realization of the necessity for the use of larger locomotives. On the Prussian State Railways (as elsewhere), owing to the continued tendency to increase the dead weight of passenger equipment, both actually and per passenger, in order to provide the better accommodations demanded by the traveling public and also the greater strength required for increasingly high speeds, the average weight per seat on passenger equipment has increased from 0.27 to 0.37 tons, or 37%, in the last 20 years. The introduction of heavier locomotives has reduced the train and locomotive mileage by eliminating many assistant engines, and also the axle mileage by the introduction of heavier wagons. The coal consumption was reduced from 60.05 tons in 1907 to 53.50 tons in 1910 per 1 000 000 ton-kilometers, in spite of the fact that the average speed was considerably increased. The ton-mileage of empties was considerably reduced during this period, and the tariffs were also reduced, yet notwithstanding this the consumption of coal for each 1 000 marks of receipts was reduced from 5.14 tons in 1907 to 4.65 tons in 1910. Mr. Hammer attributes these and other favorable results almost entirely to the use of larger locomotives and consequent increase of train loads.

In some instances in the United States, as pointed out later, the tendency to increase train loads has been carried beyond the economic limit, but these excesses (involving train loads as great as 8 000 tons) do not vitiate the general argument, that there must be a considerable increase in the Argentine to meet the necessities of modern transportation and to carry bulk freight long distances at rates which will enable the hitherto undeveloped sections in the interior of South America to enter the markets of the world, and that the necessary increase is not possible on the narrow-gauge.

Saving in Fuel.—The saving in coal consumption, made possible by the use of some of the more recent types of heavy engines, is also shown by the following quotation from a recent paper† by Mr. O. S. Beyer, Jr. In commenting on fuel, he said:

"Numerous tests and service records have revealed that large superheater Mikado locomotives which have been placed in service recently haul trains of 45 and 50% greater tonnage with the same amount of

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* *Bulletin*, International Railway Congress, March, 1913.

† Am. Inst. of Mech. Engrs., Vol. 34, 1912, p. 1301.

coal that was formerly consumed by the Consolidation locomotives they replaced. Even the coal consumption of Mallet engines with grate areas up to 100 sq. ft. has not grown in any way proportionate to the increase in their hauling capacity. Modern engines when running at shortened cut-offs over those portions of the road other than the ruling grades exhibited a still greater economy than when working on the heaviest grades. Some service tests of recently built Mikado engines on the Delaware, Lackawanna and Western Railroad clearly demonstrated these facts. Their economy in fuel consumption as compared with that of the old Consolidation type, both operating over heavy grades at full load, being 20 per cent. The economy effected over easy grades while running at shortened cut-offs was 39.3 per cent. almost twice as much. The average was 29.1 per cent."

Weight of Trains.—Table 11, showing the records of heavy trains actually hauled in the ordinary course of business in the United States, will give some idea of the present state of the art there. The records are all within the past 3 years, and the length of the run varies from about 125 miles on the low gradients of the Virginian and Pennsylvania Railways to 50 to 65 miles on the other lines with steeper gradients.

TABLE 11.

Railroad.	Type of locomotive.	Train behind tender, in tons.	Gradient. Percentage.
Denver and Rio Grande	Mallet	500	4.00
Southern Pacific	"	1 250	2.20
Canadian Transcontinental	"	1 212	2.20
" "	"	4 290	0.40
Pennsylvania Railroad		8 778	0.23
" "	Consolidation	7 644	0.20
Virginian Railroad		6 023	0.20
" "	Mikado	7 562	0.20
Lake Shore and Michigan Southern		7 433	

The last five trains carry coal or iron ore in all-steel gondolas, all of the same type and of 100 000 lb. capacity, having 85 to 100 cars in a train.

Wider Rolling Stock for Broad-Gauge.—This brings us to the consideration of possible benefits to be derived from the broad-gauge, if advantage should be taken of the extra width of track to get the same proportionate width of cars as the medium- or narrow-gauge.

Experience has shown that the width now in use on the meter-gauge, combined with the height of passenger coaches, box cars, and cattle cars, is practically at or even beyond the limit. One has only to travel

on meter-gauge lines, even where the track is well kept up, to realize that as soon as any attempt at speed is made the oscillations are quite violent, and if the track is out of line and surface, speed becomes positively dangerous.

On the standard-gauge, which is well named the normal in many countries, the happy medium appears to have been most nearly attained by proper adjustment of the size of the vehicles to the width of the track. The clearance diagrams, Fig. 3, which are those established for the Argentine, show the ratios in Table 12 between the clearances on the three gauges; and it may be noted that the clearances for medium-gauge are ample for the largest equipment in use in the United States.

TABLE 12.

Gauge.	Permissible width. Argentine.	Width of track.	Ratio.
Broad	11 ft. 1¾ in.	5 ft. 6 in.	2.0 to 1
Medium	10 ft. 10 in.	4 ft. 8½ in.	2.3 to 1
Narrow	10 ft. 6 in.	3 ft. 3⅜ in.	3.2 to 1

TABLE 13.—RATIOS BETWEEN THE ACTUAL WIDTHS OF VEHICLES AND WIDTHS OF TRACK.

Gauge.	Actual width of vehicle.	Width of track.	Ratio.
Broad	10 ft. 6 in.	5 ft. 6 in.	1.91 to 1
Medium	10 ft. 2 in.	4 ft. 8½ in.	2.16 to 1 (Freight cars only 9 ft. 4 in.)
Narrow	9 ft. 1 in.	3 ft. 3⅜ in.	2.77 to 1

The statement is often made—and at first sight apparently with a certain amount of reason—that the broad-gauge has not taken proper advantage of the additional width of gauge. It has already been shown that there would be no advantage in increasing the width of passenger coaches. Up to the present there has been little demand for freight cars of greater capacity than 100 000 lb. (45 000 kg.), but, if there were, it could be met, up to at least 20 to 30% more, without increasing the width of any type. The only possible advantage of the increased width would be that, theoretically, a reduction in cost would be possible, because the nearer the body of the car approches a square in plan the more economical it is to build, and in this the 5 ft. 6-in. gauge has a

superiority over the meter-gauge which more than compensates for the cost of the additional width of track. Between the present width of the broad-gauge rolling stock (10 ft. 6 in.), however, and the width to make it proportionately equal to the medium-gauge, say 12 ft., there

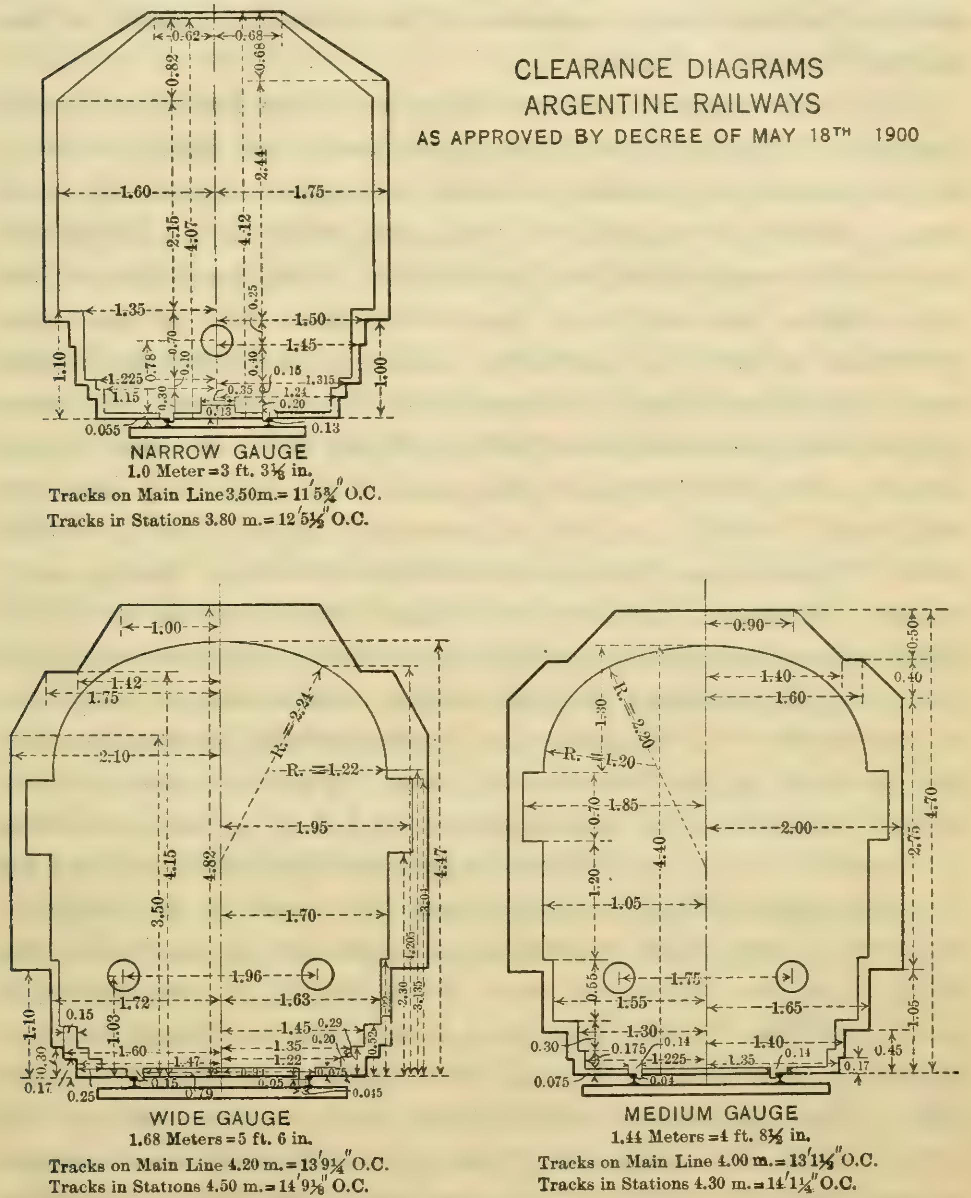

FIG. 3.

would be little saving in the cost of the vehicles, and no advantage in carrying capacity, even though the increased cost of additional clearance required for overhead structures, tunnels, etc., is practically negligible in the Argentine, as there are so few. There has recently been

designed for the Norfolk and Western Railway, and placed in service, high-sided steel gondolas of 100 tons capacity, weighing light about 65 000 lb., and having a simplified form of six-wheel truck. These cars are 10 ft. 4½ in. wide, over all, and their use indicates the possibility of a long step in advance in the operating capacity of the standard (4 ft. 8½ in.) gauge.

It has been suggested, however, that much larger locomotives would be possible if the wider clearance were allowed, and that, in view of the continuous and insistent demand for larger engines and more power to haul faster and heavier trains, this would be a most decided benefit. The average total weight on drivers, of the locomotives of the United States, increased from about 69 000 lb. in 1885 to more than 180 000 lb. in 1907, and reached a maximum of 316 000 lb. in that year. (The latest Mallet, 1912, has 550 000 lb. on drivers.) The average axle loads increased from 22 000 to 48 000 lb. in the same period, and since then have increased still more, being now, in at least one case, nearly 70 000 lb., and this on passenger engines running at high speeds (Pennsylvania Railroad, 4-4-2 with 68 000 lb. per driving axle). In view, therefore, of the general demand throughout the world for bigger types of everything, and most particularly in the transportation line, it is not surprising that the average person sees no reason for not continuing to build larger and more powerful locomotives, and that, in view of the fact that in some respects we seem to have reached the limit on the medium-gauge, the 5 ft. 6 in. looks attractive, and especially so in the additional width which might be utilized for the power plant. Reference has already been made to the statement that there is now under construction a locomotive of the Mallet type which is to have a tractive power of 160 000 lb., as compared with about 110 000 lb. for the largest now in use, thus marking a long step forward in the ability of the 4 ft. 8½-in. gauge to handle the heaviest traffic. There are some signs, however, that in some cases in the United States the length and weight of freight trains have reached or even passed the economic limit, and that there may be a reaction from the extremely heavy loadings, though by no means to reduce them to anything like the low standards of Europe or South America.

It has been pointed out* that these extremely large engines have

* *Engineering Record*, February 20th, 1909.

greatly increased the cost of bridges, and that heavy locomotives, because of the increased stresses in the track, increase the cost of maintenance. High speed, combined with heavy weights, is the cause of far more rail failures than poor steel. Lighter locomotives, shorter trains, and higher speed will give more service from the cars and more satisfaction to the shipper, who is not altogether in favor of holding trains for the last car the engine can haul.

In an editorial in a technical journal* relating to the controversy over the gauge question in Australia, the question of the advantage or otherwise of a broader gauge in the United States was examined; not, of course, with any idea of changing, but purely for the purpose of discussing what advantages there might have been had a wider gauge been adopted from the beginning, and the conclusion was that there would have been no benefit. Considering the matter further† the same journal gives the following reasons for thinking we have reached the economic limitations of the size of the locomotives and weights of trains:

Long heavy trains are only desirable if they save money.

The big locomotive and long train have been the principal factors which have enabled American Railways to pay double the wages paid in Europe and at the same time reduce the ton-mile cost of hauling freight far below that of any country in Europe.

The principal saving in operating expenses by reason of heavy, long trains is in saving in wages of train crews, which item amounts to about 12½% of the total operating cost.

Legislation and labor organizations are nullifying this saving by making the pay more or less proportionate to the weight of the train or the size of the engine.

An argument in favor of long trains has been that the cost of controlling a train is practically the same whether it be long or short, but train despatchers as well as train crews have much more trouble getting very long trains over the road in time.

Excessive length of trains may and does interfere with their prompt and efficient handling at terminals or passing sidings, causing delay to other trains.

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* *Engineering News*, December 7th, 1911.

† *Engineering News*, July 18th, 1912.

Heavy locomotives and high speed increase cost of maintenance very rapidly as they pass the normal, 125 lb. rails of open-hearth steel, with special alloys, are already being considered to meet the demands of heavy traffic, and this expense is a charge against the heavy locomotive.

Probably the most important consideration of all, however, is the item of rolling stock repairs which has increased very rapidly with the increased weight of train and power of the locomotive. A hauling power of 25 tons and upward cannot be transmitted through a long freight train without exerting enormous stresses upon draft gear, couplers, car sills, floors, etc. It is not the mere push or pull of the locomotive, but the transmission of the power in a series of waves or shocks, especially on lines with broken profiles.

This matter of the proper design of rolling stock, and the strengthening of the underframes and draft rigging, is one which is receiving considerable attention, but, of course, the raising of the general standard of efficiency in a matter of this kind is very gradual, and in the meantime there is a temporary halt.

Enough has been said, however, to show that the 5 ft. 6-in. gauge offers no advantage in allowing the use of heavier cars or trains than are now in general use or in contemplation on the 4 ft. 8½-in., and the latter is capable of much further expansion along these lines, even beyond what are to-day considered to be the economic limits.

Train Resistance.—The term "train resistance" is used to denote the combination of forces which have to be overcome to produce the movement of the train, these forces being affected by the character of the road and vehicles and, therefore, to some extent, by the gauge.

Train resistance is of two kinds: that due to the internal resistance of the train, and that due to gradients and curvature. Numerous experiments have been made to determine the amount of the former, with varying results due to the complex nature of the problem, but one fact is firmly established, that the resistance per ton of train is much less for loaded cars than for empties, and less for heavier cars than for light ones. For a train of loaded 50-ton coal or mineral cars, all of the same type, the resistance, at ordinary freight-train speeds on straight, level track, is often as low as 3 lb. per ton, whereas trains of miscellaneous cars in the United States, including a fair proportion of cars as light as 30 tons and some empties, will have a resistance as

high as from 7 to 10 lb. per ton, 7 lb., however, being about the maximum with fairly good conditions. There is no doubt, therefore, that in this respect the narrow-gauge, with its much lesser average weight per axle, is at a disadvantage when compared with the broad-gauge, but just how much it is impossible to say.

The following table shows the effect of the capacity of cars on the loads which a consolidation engine (118 tons) can haul on a 0.5% grade at 20 miles per hour. The capacity includes weight of car and load, and the tonnage is based on the formula of the American Railway Engineering Association ($R = 2.22\ T + 121.6\ C$):

20-ton cars................	1 570 tons.
30 " "	1 775 "
40 " "	1 910 "
50 " "	2 010 "
60 " "	2 090 "
72 " "	2 160 "

The resistance due to grade is the force required to lift a certain weight a certain height; its computation is an exact mathematical proposition, and is not influenced in any way by gauge.

The resistance due to curvature is probably slightly less on narrow-gauge than on broad-gauge. It is of two kinds: one is the friction of the flanges of the wheel on the rails, which is probably in direct proportion to the force required to change the direction of a body tending to move in a straight line, this being a function of the speed and weight, and having nothing to do with the gauge; the other is the slipping of the treads of the wheels on the top of the rails, and this varies with the width of the gauge. That the latter is a comparatively unimportant factor is realized when one considers that, for example, on a curve of 400 ft. radius with a central angle of 90°, the difference in the length between the outer and inner rails for 4 ft. 8½-in. gauge is 7.39 ft. and for the meter-gauge, 5.15 ft. On a curve of 1 000 ft. radius with a central angle of 30°, which is about an average curve in the Argentine, the difference between the outer and inner rails would be:

Broad	2.88 ft.
Medium	2.47 "
Narrow	1.72 "

When it is considered that about 90.4% of the alignment in the Argentine, including in this all the mountain lines, is on tangent, this additional resistance due to the narrower gauge is practically negligible. On a strictly mountain line, with very heavy and continuous curvature, this additional resistance might be considered perhaps to be equal, roughly, to the additional resistance of the smaller class of rolling stock, but, for the average line, it would be considerably less. On the other hand, locomotives for the narrow-gauge, of equal weight and power with those of wider gauge, must necessarily be longer, and, therefore, the resistance encountered by them in passing around curves would be somewhat greater than with the shorter engines for wider gauge.

Taken altogether, therefore, it seems quite safe to say that the resistance per ton of train on narrow-gauge lines is greater than on wider gauge, and the cost of operation is thereby increased, though it is hardly possible to calculate the amount.

Costs of Construction.

The reasons which are most generally advanced by the advocates of narrow-gauge for the supposed economy in first cost of construction are as follows:

1.—The use of sharper curvature is supposed to permit better adaptation of the line to the conformation of the ground.

2.—The cost of earthwork is supposed to be less:

a.—For reason No. 1.

b.—Because they are narrower.

3.—The cost of bridging is supposed to be less:

a.—Because they are to carry lighter loads.

b.—Because they are narrower, especially the masonry.

4.—The track and rolling stock are supposed to be lighter.

All these reasons are invalid except those directly affected by the width, that is to say, the width of earthwork and the length of ties; and, theoretically, it is questionable if even these are. If the bridges, track, and rolling stock are lighter, they are correspondingly less efficient, and, generally, in lesser proportion, as, for instance, a 30-lb. rail will only carry one-quarter the load that a 60-lb. rail will.

In nearly all the discussions, etc., in regard to gauge, comparisons of costs are made between two or more lines already built, or the average cost of all the broad-gauge lines of a country is compared with the average cost of all the narrow-gauge lines. Such comparisons, of course, are valueless for the determination of the effect of gauge on cost of construction. The broad-gauge lines usually are of a much heavier type of construction, have heavier rails, heavier rolling stock, and more of it, expensive terminals, etc., as they usually do more business. To give only one example of the many which have been found in looking over previous discussions, the following,* is a very fair example of the kind of argument used and information on which arguments have often been based. Speaking of Indian railways, the statement was made that broad-gauge costs 2½ times as much as meter-gauge. If the Government of India had pinned its faith to broad-gauge it would now (1906) have 7 000 miles less of railroad. The cost of line in India having been £11 775 per mile for broad-gauge, as against £4 700 per mile for meter-gauge.

In the discussion following, it was pointed out that the broad-gauge lines did more business, had heavier work, that is, greater quantities of earthwork, larger terminals, etc., and that to take only one item, that of bridging and tunnels alone, these had cost Rs. 10 792 ($3 500) per mile for the broad-gauge as against Rs. 2 149 ($700) for the meter-gauge, so that if the meter-gauge lines had been built in the same location as the existing broad-gauge and equipped to do the same business (if this were possible) they would have cost quite as much.

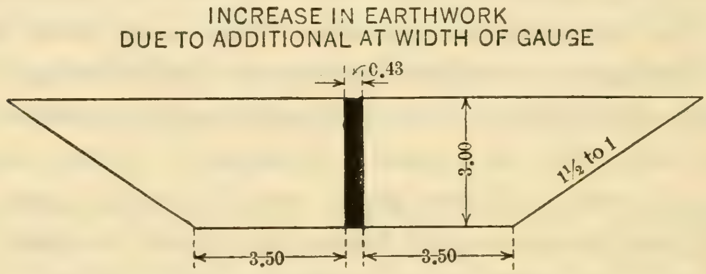
In connection with the further discussion of this paper, the following estimates were made of the difference in cost of building broad- and narrow-gauge lines in India:

Sir John Hawkshaw............	£360	per	mile	=	$1 800
Sir John Fowler...............	633	"	"	=	3 165
Sir George Bruce..............	200	"	"	=	1 000
Sir Guilford Molesworth.......	425	"	"	=	2 125
and in Victoria, Australia......	261	"	"	=	1 305
and South Australia...........	350	"	"	=	1 750

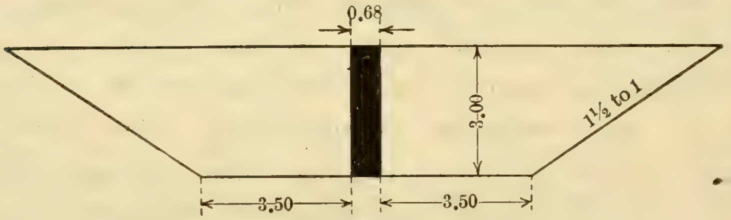
It is to be noted that these differences are somewhat less than those shown in the estimates which follow, and are more nearly what they

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIV.

might be if the calculations were made on a strictly theoretical basis, and including only the actual outlay for labor and material. In the estimates which follow (which are discussed in detail), not only has every reasonable allowance been made for any difference there is, but the same proportion is carried through the percentage of contingencies, interest during construction, etc., which tends to increase them, so that what difference they show is the maximum possible.



Meter Gauge Cuts 7.00 Wide Area = 34.5 Sq.M.
4′8½″ Gauge Cuts 7.43 Wide Area = 35.8 Sq.M.
Medium 3⁷⁄₁₀% Greater than Narrow



Meter Gauge Cuts 7.00 Wide Area = 34.5 Sq.M.
5′6″ Gauge Cuts 7.68 Wide Area = 36.5 Sq.M.
Broad 5⁸⁄₁₀% Greater than Narrow

FIG. 4.

The only satisfactory method of determining the effect of gauge on cost of construction is to calculate what effect a difference of gauge would have, assuming the same conditions.

We have already seen that, within the limits of practical railroading for the gauges under consideration, curvature and gradients are not affected by gauge, therefore the location would be the same for lines of either gauge through the same country, for, in order to make a fair comparison, we must assume the road designed in either case to do the same business.

Taking each item of cost, therefore, we find:

Right of Way, Station Grounds, Terminals, etc.—These are not affected, the difference in width of earthwork being only 0.70 m. between meter- and broad-gauge, so that the width of right of way is not changed. If the tracks are spaced closer in terminals for narrow-gauge, they must be longer to hold the same tonnage.

Clearing and Grubbing.—There is no difference.

Earthwork.—This item is the one most affected by gauge, but the proportion is very small, as will be seen by the diagram, Fig. 4, showing, in the case of a cut or fill 3 m. deep, 4% more for medium over meter and 6% more for broad over meter. In making the calculations in the following tables, for lines with very heavy earthworks, an additional allowance has been made to cover the extra width in mountain work with steep transverse slopes, where the additional width involves somewhat more earthwork.

Bridging and Culverts.—The width of masonry is increased somewhat by reason of increased width of banks, but the amount is very small, about 2 to 4% for abutments, and from 0.2 to 1% for culverts, the end walls of the latter being the same, with only an increase of from 1 to 2 ft. in the length of the barrel. The steel work will be the same.

The latter assertion may be questioned, in view of the fact that much narrow-gauge rolling stock is lighter than medium or broad-gauge, but in order to avoid too much complication, it is assumed for purposes of this comparison that the same number of trains will handle the business, therefore the same weight of engine will be assumed. If any other assumption is made, that is, that heavier engines are used for the wider gauges, that means less trains and less cost of operation, which would compensate for the additional weight of the steel. The total cost of the actual bridging, omitting abutments, etc., is such a small proportion of the total cost of the railroad that any slight difference in this item would be very small.

Track.—Rails and fastenings will not be affected. The cost of rails is taken at $30 at Buenos Aires with about $150 per mile added for freight. The length of the cross-ties is affected directly as the width of the gauge, but as the function of the sleepers is to distribute the load, as well as to hold the rails in position, there would only be as many required for broad-gauge as would give the same bearing

area on the soil or ballast as those of the narrow-gauge, and, therefore, theoretically, there would be no difference in this item. Practically there is, however, but with the result that a better track is obtained. The number of the sleepers has been assumed, therefore, at 1 400, 1 500, and 1 600 per kilometer (2 254, 2 415, and 2 576 per mile) for broad-, medium-, and narrow-gauge, respectively.

Quebracho ties are used almost exclusively on the Argentine railways; they are very hard and durable, and, owing to their high cost, are spaced rather farther apart than is usual in the United States. The average price, including freight, at Santa Fé, a fairly central point, for the area under consideration, is:

5 ft. 6-in. gauge	\$1.90 each	= \$2 660 per km.	\$4 283 per mile.
4 ft. 8½-in. gauge	1.60 "	= 2 400 " "	3 864 " "
Meter-gauge	1.20 "	= 1 920 " "	3 091 " "

Laying and Surfacing.—This has been assumed to be somewhat more for medium- or broad-gauge, on account of the greater width, but, although it is a little doubtful whether this is really warranted, it is taken in order to be on the safe side.

Ballasting.—This is assumed to be the same for all gauges, for, in order to carry the same loads, a greater depth of ballast is required for narrow-gauge, where the loads are concentrated, than for the wider gauge. Then, again, the impact is greater on narrow-gauge, due to the greater height in proportion to the width of the rolling stock.

Yards and Sidings.—These are taken at a percentage of the cost of the track in each case. Here again, theoretically at least, there should be no increase in cost due to gauge, for, although the broad-gauge track costs more per linear foot, there is not so much of it required, broad-gauge trains of the same net tonnage requiring a lesser length of track than narrow-gauge trains. As, however, on lines of comparatively light or medium traffic the yards and sidings are likely to be made the same length in any case, the increase is made as shown.

Cattle Guards.—These will be slightly wider for wider gauge.

Miscellaneous Structures, etc.—Fences, Track Signs, Buildings of all kinds, Water Supply, Telegraphs, Docks and Wharves, General Expenses of Construction, Engineering and Legal Expenses, would not be changed.

Contingencies, Expenses of Promotion, Interest During Construction.—These are figured at percentages of the totals.

Equipment.—For the same amount of traffic, the equipment would probably cost slightly less for the wider gauge on account of the larger capacity of the cars, but, assuming as nearly as possible the same weight of engines, etc., this would not be a great deal.

Tables 15, 16 and 17 show the differences due to gauge alone on lines of light, medium, and heavy earthwork and bridging, and assuming the same traffic and carrying capacity on all. It will be noted that for lines of light and medium construction there is very little difference in the percentage of increase, what difference there is being a less percentage where the work is heavier, medium-gauge averaging 6% more than narrow-gauge and broad-gauge 10% more than narrow. For very heavy lines, that is, mountain lines with steep transverse slopes, there should be some additional increase in earthwork due to the increased width. This has been allowed for, and in this class of lines the medium-gauge might be assumed to cost 11% more and the broad 16% more than the narrow.

The summary of the totals is given in Table 14, in which the percentage is the increase above the cost of narrow-gauge.

TABLE 14.—Summary of Costs.

Gauge.	Cost per Mile, in Gold Dollars.			Cost per Kilometer, in Gold Dollars.		
	Light.	Medium.	Heavy.	Light.	Medium.	Heavy.
Narrow........	$29 772	$54 760	$92 573	$18 458	$33 951	$57 395
Medium.......	31 581 + 6%	57 920 + 6%	102 892 + 11%	19 580	35 910	63 793
Broad..........	32 861 + 10%	60 430 + 10%	107 553 + 16%	20 374	37 467	66 683

It may be noted that the assumed increases in Table 14, which the writer believes to be the maximum or outside amounts, are not more than the usual allowances for contingencies in work of this magnitude.

Cost of Conversion.

The following estimate of the cost of changing the gauge, although not based on detailed surveys or calculation of quantities, is believed to represent average conditions, based on observations by the writer after an inspection of practically all the meter and standard-gauge

lines of the Argentine, as well as of a considerable proportion of the broad-gauge lines.

It is believed that it can be safely assumed that the cost of conversion, including a liberal allowance for rolling stock, will not exceed the figures given, and will probably be less. It is to be noted, however, that these estimates do not include any betterments, such as heavier rails, renewing bridges, etc., or any other works of improvement which it might be desirable to carry out at the time the gauge would be changed; but they do include an allowance for all new sleepers. It may also be noted that most of the lines which would be changed first are through easy country, and the cost would be less than the average.

TABLE 15.—Cost of New Construction.

Light Earthwork and Bridging.

	Cost per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½ in.	5 ft. 6 in.
Right of way. station grounds, terminals	$40	$40	$40
Clearing	200	200	200
Earthwork	4 000	4 310	4 570
Bridges	2 000	2 150	2 300
Culverts	2 000	2 150	2 300
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards. fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply etc	2 000	2 000	2 000
Telegraph and telephone	500	500	500
Docks and wharves	50	50	50
General expenses and administration	300	300	300
Engineering	1 000	1 000	1 000
Legal expenses	250	250	250
	$23 280	$24 940	$26 140
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% } 15%	3 492	3 741	3 921
	$26 772	$28 681	$30 061
Equipment (rolling stock, etc.) assumed for same amount of traffic	3 000	2 900	2 800
	$29 772	$31 581	$32 861
Per kilometer	$18 458	$19 580	$20 374

TABLE 16.—COST OF NEW CONSTRUCTION.
MEDIUM EARTHWORK AND BRIDGING.

	COST PER MILE, IN GOLD DOLLARS.		
	Meter.	4 ft. 8½ in.	5 ft. 6 in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	250	250	250
Earthwork	18 450	19 430	20 260
Bridges	5 000	5 500	6 000
Culverts	5 000	5 300	5 600
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc	2 500	2 500	2 500
Telegraph and telephone	550	550	550
Docks and wharves	50	50	50
General expenses and administration	400	400	400
Engineering	1 300	1 300	1 300
Legal expenses	270	270	270
	$44 750	$47 580	$49 850
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% } 15%	6 710	7 140	7 480
	$51 460	$54 720	$57 330
Equipment (rolling stock, etc.) assumed for same amount of traffic	3 300	3 200	3 100
	$54 760	$57 920	$60 430
Per kilometer	$33 951	$35 910	$37 467

The following* shows the actual cost of changing certain lines in India from meter to 5 ft. 6 in. The costs include rolling stock and, in at least one case, the Salt Branch of the North Western, the cost of changing from 41-lb. to 75-lb. rails, and in the last case there were "heavy charges for bridge renewals and heavy permanent way".

COST OF CONVERTING METER-GAUGE TO 5 FT. 6-IN. IN INDIA.

			Per mile.	Per kilometer.
Salt Branch of North Western,	50.0	miles	£2 867 = $14 350	= $8 600.
Nagpur-Chattisgarh,	145.2	"	£2 583 = $12 915	= 7 750.
Kotkapura-Firozur,	27.9	"	£1 472 = $73 60	= 4 415.
Gudur to Nellore,	24.2	"	£4 125 = $20 625	= 12 374.

* From *Minutes of Proceedings*, Inst. C. E., 1906.

TABLE 17.—COST OF NEW CONSTRUCTION.
HEAVY EARTHWORK AND BRIDGING.

	Cost per Mile, in Gold Dollars.		
	Meter.	4 ft. 8½ in.	5 ft. 6 in.
Right of way, station grounds, terminals	$40	$40	$40
Clearing	300	300	300
Earthwork	39 490	44 500	46 000
Bridges	10 000	12 000	13 000
Culverts	10 000	11 000	12 000
Track: Rails and fastenings, 70-lb. rail	4 000	4 000	4 000
Ties	3 090	3 860	4 280
Laying and surfacing	800	900	1 000
Ballasting, half earth, half stone	850	850	850
Yards and sidings, 20% of amount for rails, ties, laying, and surfacing	1 750	1 920	2 030
Cattle guards, fences, track signs	450	460	470
Buildings, shops, stations, section houses, water supply, etc	3 000	3 000	3 000
Telegraph and telephone	600	600	600
Docks and wharves	50	50	50
General expenses and administration	500	500	500
Engineering	1 800	1 800	1 800
Legal expenses	300	300	300
	$77 020	$86 080	$90 220
Contingencies, 5%; Expenses of promotion, 2%; Interest during construction, 8% — 15%	11 553	12 912	13 533
	$88 573	$98 992	$103 753
Equipment (rolling stock, etc.) assumed for same amount of traffic	4 000	3 900	3 800
	$92 573	$102 892	$107 553
Per kilometer	$57 395	$63 793	$66 683

The author of the paper, Sir Frederick Robert Upcott, who gave these costs, estimated that the cost of changing the existing meter-gauge lines of India to 5 ft. 6-in. would be £2 500 ($12 500) per mile. or about £1 500 ($7 500) per kilometer.

It is stated* that the cost of changing the gauge of the lines in Queensland and West Australia from 3 ft. 6-in. to 4 ft. 8½-in. will be $111 000 000, or an average of $16 600 per mile ($10 000 per km.) for the 6 662 miles to be changed.

It has also been stated that the estimated cost of changing the gauge of the railways in Japan will be some $125 000 000 for about

* *Railway Age Gazette*, July 11th, 1913.

5 000 miles, or $25 000 per mile ($15 000 per km.). This seems high, but, of course, the topography of the country is quite rugged and mountainous and the railways have many large bridges and tunnels.

TABLE 18.—Estimate of Cost of Changing 1 Mile of Meter-Gauge to Medium- or Broad-Gauge.

Assuming Average Conditions on the Meter-Gauge Lines of the Argentine.

		Gold Dollars.	
		Medium.	Broad.
Right of way, station grounds, terminals, etc., $10 000 for additional terminal land for every 300 miles		$33	$33
Earthwork: Say, 5% additional for medium and 7% for broad on assumed present average cost of about $20 000 per mile		1 000	1 400
Bridges and culverts: lengthening		2 000	2 500
Track: Medium gauge:			
2 400 new sleepers at $1.60	$3 840		
New fastenings	250		
Taking up, relaying and surfacing	1 500		
		5 590	....
Broad-gauge:			
2 600 new sleepers at $1.90	$4 940		
New fastenings	250		
Taking up, relaying, and surfacing	1 500		
		....	6 609
Yards and sidings: 20% of main line		1 118	1 338
Cattle guards, nominal		15	15
Fences, track signs, etc., no change		....	....
Buildings, shops, stations, section houses, turntables, water supply. Will only involve changing tracks and spacing in shops, turntables, engine-houses, and running sheds, say $200 000 for every 300 miles		630	700
Signals and interlocking		75	75
Telegraph and telephone, no change		....	....
General expenses and administration		100	100
Engineering		250	250
Legal expenses		25	25
Interest during construction		600	700
Contingencies		600	700
		$12 036	$14 526
New rolling stock, say $5 000 per mile. Credit old rolling stock 3 500 per mile		1 400	1 600
Per mile		$13 436	$16 126
Per kilometer		8 330	9 998

Cost of Operation.

Various statements have been made in regard to the relative costs of operation and maintenance of lines of different gauges, almost invariably however, so far as the writer has been able to discover, based on comparisons of the costs on two existing lines or groups of lines. Such comparisons are not of great value, for no matter how generally similar two lines may be, or seem to be, the inherent differences in

physical condition, amount and character of traffic, rolling stock, management, policy, etc., will be considerable. An attempt was made by the writer to analyze the details of the operating costs of certain lines, and from this to form some idea of the actual effect on each item of a change or difference in the gauge, but it was found that there was so little information available that the result would be too much in the nature of a pure guess to be of any value, and as there seems to be no really good way of determining what actual effect a difference of gauge would have on the operation and maintenance costs, such comparisons between the results on lines of different gauges in various parts of the world as are available are given for what they are worth, followed by some discussion of the effect of gauge on maintenance of permanent way and an analysis of the possible economies by the use of heavier rolling stock, etc., both in maintenance of equipment and costs of operation.

Indian Railways.—In regard to the railways of India, the following statement was recently made by Sir Guilford Molesworth, showing that although the train-mile costs on the narrow-gauge lines of that country were lower than on the broad, the net ton-mile costs were higher.

Taking the East India Railways (5 ft. 6-in.) as 100, the costs on the following narrow-gauge lines were:

	Per train-mile.	Per ton-mile.
Assam Railway..................	71	315
South Maharatta................	56	280
Burma Line....................	64	290

A statement prepared by Sir Frederick Upcott, giving the comparative statistics of eight of the principal broad-gauge and eight of the principal narrow-gauge lines, for 1903, showed that, though the earnings on the narrow-gauge lines gave practically the same returns on the capital invested as on the broad-, the average charges were:

	Cents, in gold.	
	Broad.	Narrow.
Per passenger-mile	0.42	0.40
Per freight-mile	0.92	1.14

and the cost of carriage of freight per ton-mile was:

0.5 cent for the broad-gauge.

0.6 " " " narrow-gauge.

It is interesting to note, here, that the average weight of trains on the broad-gauge was 167 tons, as compared with 88 tons for the narrow-gauge.

Australian Railways.—The ton-mile costs in Australia are not available, but the train-mile costs for 1910-11 were as follows:

Broad	$1.20
Medium	1.09
Narrow	1.32

These figures, taken together with the fact that the net earnings per mile of the medium-gauge lines were double those of the narrow-gauge, indicate the inferiority of the earning power of the latter.

Argentine Railways.—In the Argentine the average operating costs of all lines for 1909 were as follows:

	Broad.	Medium.	Narrow.	Broad.	Medium.	Narrow.
	Per train-kilometer.			Per train-mile.		
Dollars, gold....	$1.006	$0.755	$0.876	$1.620	$1.216	$1.410
	Per ton-kilometer.			Per ton-mile.		
Cents, gold.....	0.928	0.957	0.967	1.494	1.540	1.557

The ton-mile (ton-kilometer) used for these figures is that of the "peso util" or "paying weight" transported, and includes passengers, with ordinary baggage, calculated at 100 kg. (220 lb.) each, excess baggage, and express, as well as freight; but, as the proportion of revenue from passengers, baggage, and express is much greater on the broad-gauge lines, being 31% of the total as compared with 19% for the narrow-gauge, the comparison is hardly fair to the broad-gauge. It shows, however, the same tendency to low cost per ton-mile for the wide-gauge, although the train-mile costs, due to the heavier trains hauled, are higher.

It is to be noted, as previously stated, that the medium-gauge lines, do not offer any real comparison on account of local conditions, and this applies also to the ton-mile costs on these lines, which are given later.

Taking the figures as they stand, and using the number of ton-kilometers for 1909, they show a saving in operating costs of a little more than $100, gold, per kilometer ($160 per mile) per annum in favor of the broad-gauge, which, of course, is not very large. In discussing these costs, however, consideration must be given to the essential difference and physical characteristics of the broad- and narrow-

gauge lines at this time (1909). The narrow-gauge lines of the Argentine were then doing a fair amount of business, as more or less local lines, that is, none of them reached the National Capital nor did they partake of the character of trunk lines. The Central Cordoba was then building to Buenos Aires, and now (1911) all the narrow-gauge lines realize that, if they are not to be entirely crowded out, they must get in close touch with the National Capital in order to stay in business, and spend large sums for capital account to put their lines in condition even to meet reasonably the competition existing or to be expected. The narrow-gauge lines were then capitalized at about $28 000, gold, per km., as compared with $42 000, gold, for the broad-gauge, but they are now faced with the necessity of capital expenditure which will bring them well up toward the capitalized value of the broad-gauge. A large part of the capital of the latter is invested in the expensive Buenos Aires terminals, and other facilities at all points on their lines, which they have found it necessary to have in order to do their business, which the narrow-gauge lines have yet to acquire and develop, and on the capital expenditure for which they must earn the interest. It is also quite certain that many of the narrow-gauge lines would have to be double-tracked in order to handle the business.

If it could be assumed that the foregoing figures would hold good for any amount of business to be hauled, they would not present a very formidable argument for the broad- as compared with the narrow-gauge, but all railroads, as well as all other enterprises in any country as progressive as the Argentine, must grow with the country or be crowded out, and this is what is beginning to happen to the narrow-gauge lines. The business coming to them is more than they can handle economically with the plant they have, because it tends to cause congestion.

The need of adequate facilities for doing business and the increased cost of operation due to their lack when the amount of business offered passes a certain limit, was shown in 1906 in the United States, when many of the railroads of the country suffered serious losses because of the congestion caused by the large amount of freight offered for transportation which they had not the facilities for handling.

The real test, of course, is the final net earnings on the investment, and here the broad-gauge lines, in spite of their heavier capitalization,

are a long way ahead, with average earnings of 5.66% on the capital invested, as compared with 2.41% earned on the capital invested in the narrow-gauge lines, so that it is evident that the ton-mile figures do not tell the whole story.

Taking freight alone, on the basis of these 1909 figures, the receipts per ton-mile make a still better showing in favor of the narrow-gauge, the averages being:

	Freight receipts. Ton-kilometer.	Ton-mile.	
Broad,	1.17	1.95	cents, gold.
Medium,	1.26	2.10	" "
Narrow,	1.05	1.75	" "
Average for all lines,	1.15	1.92	cents, gold.

It may be noted, also, that the rates on the Central Cordoba and Santa Fé lines are the lowest of any lines in the country, and it is these which tend to lower the average for the narrow-gauge lines, but it is believed that the explanation given above shows the reason for the apparent cheaper cost of operation, or, rather, lesser rates on these two narrow-gauge lines. It may also be added that the property of neither has been kept in first-class physical condition, and, as already stated, their net earnings show a much lower percentage of the capital invested than do the broad-gauge lines. The Government lines, also, which are narrow-gauge, are operated at a loss. If the freight charges on these lines were fixed to give the same return on the capital invested they would, of course, be much higher.

Comparison of Ton-Mile Costs.—The average receipts per ton-mile of the Argentine railways in 1909 are given above. The average for the United States in 1912 was 0.85 cents. On 70% of all the railroads the average was less than 1.0 cent per ton-mile, and the lowest average rate was 0.48 cent per ton-mile.

The actual cost of moving freight on the Bessemer and Lake Erie Railway in 1909 was stated to be* 0.230 cent per ton-mile; this included terminal charges on a road only 144 miles long; adding interest on the investment would bring the total cost to 0.280 cent per ton-mile.

Maintenance of Permanent Way.—In regard to maintenance, it seems quite evident that this would be considerably more on narrow-

* *Engineering News*, April 21st, 1910.

gauge lines having any large amount of traffic, and especially if it was desired to run passenger trains even at reasonably fast speeds.

If it were possible to build and maintain a track without inequalities, or if the inequalities of the line and the forces tending to produce oscillation could be reduced in the same proportion as the width of the gauge, the cost of maintenance might be considered as proportional to the width, but these conditions are unattainable.

The narrower the gauge the greater is the angle through which the vehicle is canted laterally through a certain depression or elevation of one of the rails, and the greater, therefore, is the inequality produced in the loads on the springs on opposite sides, and, consequently, on the two rails also, so that the narrower the gauge the greater the extent of lateral oscillation to which any given inequality in the line will give rise, and this is a point of especial importance in districts where, from the variations of climate or other influences, the permanent way is likely at times to get more or less out of repair.*

It is to be noted, also, that the use of stone ballast is almost indispensable on narrow-gauge lines, in order to have track on which speeds of more that 30 miles per hour are safe, and that the depth of the ballast must be increased in order to distribute properly the loads which, in the narrow-gauge, are concentrated on a smaller area.

A few years ago it was quite generally thought that the principal function of the ballast was to afford proper drainage for the track, though even as long ago as 1887-90 experiments, which were conducted in Germany by Railroad Director Schubert, had indicated the value and necessity of ballast for the proper distribution of the loads. In view of the great increase in axle loads in recent years, an extended series of experiments was made in 1908-10 by the Pennsylvania Railroad to obtain further light on this phase of the subject.† These experiments showed quite clearly and conclusively the part played by ballast in the distribution of the loads, and the necessity of a sufficient depth to take care of this. It is quite certain, therefore, that the concentration of loads on lines of narrow-gauge, both by reason of the smaller bearing area of the sleepers and the additional impact due to the higher center of gravity and greater oscillation, must be taken care

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* See also p. 566 *et seq.*

† *Proceedings*, Am. Ry. Eng. Assoc., 1912.

of by an increase in depth of ballast, if equally good results are to be obtained.

Maintenance of Equipment.—It is quite difficult, if not impossible, to make any estimate of the effect of gauge on the cost of maintenance of equipment. It is quite sure that the cost of maintenance of both locomotives and wagons is not proportionate to either their size or capacity, that is to say, it does not cost twice as much per annum for the maintenance of a 200-ton engine as it does to maintain a 100-ton engine; two 25-ton cars will cost more to maintain than one 50-ton car doing the same work. The passenger equipment for narrow-gauge has only about three-quarters of the carrying capacity of the broad-gauge for vehicles of the same length, so that, to do the same amount of business, there will be a larger number of vehicles, larger proportion of dead weight per passenger, and, therefore, a larger cost of maintenance per revenue unit.

If it could be assumed that all or even the greater proportion of the business of the Argentine, if carried on broad-gauge lines, could be handled by locomotives and in freight cars of twice the capacity of those now in general use on the narrow-gauge, one could say at once that the cost of maintenance of equipment would be greatly lessened thereby. This, however, is not the case, and there are many locomotives and cars in use, and of a useful size, on the narrow-gauge, equal in capacity to many on the broad-, and it is only a certain proportion of the business that can be handled to better advantage in cars and by locomotives of larger capacity.

Taken altogether—passenger cars, freight cars, and locomotives—it seems quite sure than the cost of maintenance of equipment would be more on narrow-gauge lines than on the broad-gauge, by reason of the larger number of vehicles required to handle the same amount of business. Just how much this item would amount to it is not possible to say, though the figures in Table 19 may give some idea.

Effect of Larger Rolling Stock Units on Costs of General Repairs.—Assuming quite conservatively that half the business could be handled on the broad-gauge by equipment having 50% greater capacity than that now in use or possible on the narrow-gauge, then the number of units of rolling-stock would be reduced one-sixth. The cost of repairs on the larger units would be somewhat greater, but not propor-

TABLE 19.—COST OF MAINTENANCE OF LOCOMOTIVES AND CARS IN 1909 ON THE NARROW-GAUGE LINES OF THE ARGENTINE.

	Number in service.	Repairs and Maintenance.	
		Total cost.	Cost, each.
Locomotives	500	$1 051 332	$2 102
Passenger coaches	599	258 477	430
Baggage cars	402		
Freight cars	13 540	683 175	47
Service cars	540		
		$1 992 984	

tionately so, say, roughly, only 50% of the increase. That is, for example, taking the case of the locomotives, 250 would remain as at present, each costing $2 100 a year for repairs, the other 250 would be replaced by two-thirds of the number, each unit having 50% greater capacity and costing 25% more for repairs, that is, there would be 167 each costing $2 625 for repairs per annum. The total cost of repairs would then be:

Locomotives	250 ×	$2 102 =	$252 500
	167 ×	2 625 =	438 375
Passenger coaches	300 ×	430 =	129 000
	200 ×	537 =	107 400
Freight cars, etc.	7 240 ×	47 =	340 280
	4 826 ×	59 =	284 734
			$1 825 289

This shows a saving of $167 695 per annum, equal to about $23 per km. of line, with the business of 1909. With a live policy of operation, the building of branches to develop the existing main lines, it is not unreasonable to assume that, by 1919, say in 10 years, the business will be increased to three times the present amount, and a large proportion of it will need to be handled in heavy trains.

There would be all the new mileage on which there would be a similar saving to that just shown, the new mileage would bring as much business to the old lines as these now have, and the natural increase in the country adjoining the old lines might be at least equal to the business now, so that on this item of repairs alone there would be a saving of from $70 to $100 per km. on the present lines, and at

least \$20 to \$30 on the new lines, or, say, an average of \$50 per km. per annum on the whole system then in operation.

It is to be noted that if the change of gauge is brought to pass, the new equipment, if properly selected, would show a much smaller cost of repairs, as it would be better adapted to the requirements of the service. There would be also, without going to extremes, much of it of double the average capacity of the present narrow-gauge rolling stock, instead of only 50% greater as just assumed, so that the foregoing estimate is quite conservative.

Effect of Small or Large Cars on Train Resistance, Loading, Etc.—The effect of small cars on train resistance has already been pointed out.* Their effect on the cost of operation is indicated by the following:

Assume an engine of the Mikado (2-8-2) type:

Total weight of engine and tender..	246 600 lb.
Weight on drivers................	110 500 "
Maximum tractive power..........	28 900 "

This is about as large an engine as can be built for meter-gauge of a type which will give large power capacity with ability to make fairly good speeds with heavy trains.

Engines of the Mallet or articulated type are built of much greater power, but, as that type only operates efficiently at very low speeds and is best adapted to sections of heavy gradients, it does not afford as good a basis for comparison for general conditions, in fairly easy country, similar to that found in the Argentine, where much of the freight must be moved quickly, as does the Mikado type.

The tonnage-rating formula of the American Railway Engineering Association is

$$R = 2.22\ T + 121.6\ C.$$

in which

R = Total resistance on level tangent, in pounds,

T = Total weight of train behind the tender, in tons,

C = Number of cars.

This assumes good, fair rolling stock and track, and that the resistance does not increase at ordinary freight train speeds between 7 and 35 miles per hour.†

* p. 590.

† *Proceedings*, Am. Ry. Eng. Assoc., 1910.

Using this formula and applying it to trains of fully-loaded cars on a line with grades of 0.6%, it is found that the engine assumed, when working to full capacity (*i. e.*, exerting the same power in each case), can haul the loads shown in Table 20.

TABLE 20.

	No. of cars.	Net tons per car.	Tare per car, in tons.	Gross tons.	Net tons of freight.
Broad-gauge cars	24	50	18	1 632	1 200
Standard-gauge cars	24	50	18	1 632	1 200
Meter-gauge cars	34	33	13	1 564	1 122

This is an increase of 78 net tons per train, as between meter- and standard-gauge. The formula deduced by Professor Schmidt, of the University of Illinois, from the results of extensive experiments made to determine the influence of car weight on train resistance, shows quite similar results, though even less favorable to the lighter cars.

The average receipts per ton of freight per kilometer in 1909 were 1.15 cents, gold. For the train under consideration, therefore, the average increased receipts for the same operating costs per train-kilometer would be 89 cents, gold, or, with an average goods traffic of one train daily each way, $1.78 per km. per day, or about $500, gold, per km. ($830 per mile) per annum.

It is stated that some of the traffic in the Argentine can be handled better in cars of small capacity. Assuming that half the business is handled in small cars on which there is no saving, there would still be an increased income of $250, gold, per km. per annum, due to the use of the larger cars.

Assuming, then, that an average kilometer of broad-gauge line costs from $3 000 to $5 000, gold, more to build than meter-gauge, the investment is warranted from this standpoint alone, and, taking the figures at the end of the paper, the change of gauge of the existing lines would also be warranted.

The foregoing takes no account of the further saving which may be effected on the wider gauge by the use of larger engines than are at all possible on meter-gauge, thereby further increasing the size and weight of trains and, consequently, obtaining lower operating costs, nor the fact that the speed of passenger trains is limited on the narrow-

gauge, both on account of the size of the engines and the lesser stability of trains.

Economy in Increased Train Loads by Reduction in Number of Trains.—The possibilities of the use of the larger cars and heavier engines depends, of course, to a large extent, on the kind of traffic to be handled. Such articles as cereals, cattle, timber and other forest products can all be handled to better advantage in large cars. The tonnage of these articles handled on the narrow- and medium-gauge lines of the Argentine in 1909 is shown in Table 21.

TABLE 21.

	Narrow.	Medium.
Cattle	219 631	232 680
Wool, hides, etc	40 864	38 278
Cereals	955 448	294 659
Sugar	223 789	3 528
Minerals	35 500	....
Construction material*	2 153 312	140 360
Firewood	768 828	30 045
Charcoal	302 881	33 495
	4 700 253	773 045

* Includes forest products.

That is, about 5 500 000 tons, out of a total of 8 500 000 tons, were handled on these lines, or 65% of the total. The average haul on all freight in 1909 was about 180 km., and the total length of lines about 9 000 km. It may be noted, also, that with the opening of the through connection to Paraguay and the extension of a railway line through Missiones, the shipment of timber suitable for building purposes from these northern points to the south is likely to assume large proportions, and this will all be long-haul business which must be handled cheaply to make it move.

The Mikado engine previously referred to can haul, say, 1 000 tons of net load on 0.6% grades. The largest engine yet built for narrow-gauge, the Mallet (articulated) engine previously referred to, might haul, say, 2 000 tons net load. There are, however, on some parts of these lines, especially in the southern section of the Central Cordoba and all through the Provinces of Entre Rios and Corrientes, grades of 1.0% or heavier. Taken altogether, therefore, considering the grades, the fact that the big articulated engine is not a useful type for

general use, and that a certain number of engines of the Mikado type must be kept in use for general freight services, it seems safe to assume that, even under the best of conditions on the narrow-gauge lines, the heavy trains will not exceed an average capacity of 1 500 net tons, whereas on the broad- or medium-gauge this could be doubled.

The operation of such extremely heavy trains, however, is not a practical proposition in the Argentine at present, nor is it likely to be in the immediate future, though the increasing distances to the new areas constantly being opened up will continually increase the tendency in this direction.

In referring to trains of this weight, it must not be assumed that the very much less average weight of trains has been lost sight of, the average net train loads in the Argentine in 1909 having been:

Broad	327	tons.
Medium	198	"
Narrow	211	"
All lines.......	293	"

The average train loads in the United States in 1910 for all lines was 380 tons, and in Group II, comprising the States of New York, Pennsylvania, and New Jersey, 502 tons, yet there are records of single trains of more than 8 000 tons gross, and the great reduction in the cost of transportation, which has been obtained in the United States in spite of the high cost of labor, has been brought about by the use of heavy trains for bulk traffic which can only be moved profitably at low rates.*

For the purpose of this argument, therefore, and to get some idea of the possible savings due to the reduction in the number of trains, it is entirely reasonable to assume that the practical train loads for the broad-gauge may easily be one-third greater than on the narrow-gauge, or say, 2 000 net tons, as compared with the possible maximum of 1 500 tons on the narrow-gauge.

Taking into consideration the growth of the country and a continuance of the railway development, it might be assumed that within a few years the amount of this class of traffic on the lines under consideration will be doubled to, say, 12 000 000 net tons. On the medium-gauge, with train loads of 2 000 net tons, this would mean 6 000 trains,

* See also p. 581.

and the narrow-gauge would require 8 000 trains. The average haul on this traffic is now 180 km., and the length of haul is practically certain to increase. An average engine division is about 150 km., and, considering the almost certain increased length of haul, one should add, say, one-third more to the number of trains (that is, train runs handled by one crew), making the numbers 8 000 and 10 667.

The present actual operating cost, that is, cost of coal, lubricants, train crews, repairs to rolling stock, etc., is about 50 cents, gold, per train-kilometer, of which about one-third is for fuel and water. This, however, is with trains very much lighter than those under consideration. With heavier trains the fuel and water would be increased, approximately, directly in proportion to the weight of the train. Repairs would be affected somewhat more, but not in proportion.

The average present train load to which these figures apply is about 500 tons. So taking this cost of 50 cents, of which, say, 15 cents is for fuel, water, and lubricants, and multiplying these items by three, and increasing the remainder by 25%, gives the following cost per kilometer per train of 1 500 tons:

Fuel, etc.....	15 cents	× 3	= 45	cents.
Other items..	35 "	+ 25%	= 44	"
Total.................			89	cents per train-kilometer.

In increasing the weight of the train from 1 500 to 2 000 tons, the other items would not be affected as much as by the increase as above from 500 to 1 500. Fuel might be increased 33%, though this is doubtful,* and the other items hardly at all, but, say, 10 per cent. We have, therefore, for a train of 200 tons:

Fuel, etc.....	45 cents	+ 33%	= 60	cents.
Other items...	44 "	+ 10%	= 48	"
Total.................			108	cents per train-kilometer.

Taking the average run as 150 km., the cost per train would then be:

For trains of 1 500 tons, 150 km. × 89 cents = $133.5 per train.
" " " 2 000 " 150 " × 108 " = 162.0 " "

and there would be:

On the narrow-gauge	10 667	trains at	$133.5 =	$1 423 912
" " medium-gauge	8 000	" "	162.0 =	1 296 000
				$127 912

*See p. 584 *et seq.*

or equal to about $14.20 per km. of line, due to the reduction in the number of trains.

In the foregoing calculation it has been assumed that the costs of fuel, water, and lubricants increase directly in proportion to the weight of trains, but most of the larger modern engines have shown a marked economy in this respect, and the coal consumption, etc., has been quite a little less, proportionately, to the work done.

It may be noted that, in making these assumptions, it is not considered that there will be any additional costs on the standard-gauge, due to the operation of the heavier trains, which in some cases have to be considered. It is believed that a 1 500-ton train will be quite as destructive to the track on the narrow-gauge as a 2 000-ton train on the medium-gauge, and that the time factor will not enter into the question, as an engine designed to haul a 2 000-ton train on standard-gauge will make as good time and with less effort than will a narrow-gauge engine designed to haul 1 500 tons, on the same grades and alignment.

The Necessity of an Efficient Transportation Machine.—The possibility of the competition of other broad-gauge lines or of river transportation has been referred to, and it must be kept constantly in mind that cheap, but at the same time efficient, transportation is a necessity, both for the development of the country and on account of the river competition.

The following quotation from a recent review of the general railroad situation in the United States by M. C. Colson, an eminent French engineer, shows how cheap railroad service has affected transport by water, and has a direct bearing on the necessity of establishing a railway system in the Argentine, which will accomplish similar results there. It hardly seems possible with the narrow-gauge to approach the low cost of transporting bulk freight which can be achieved on the medium- or broad-gauge.

After pointing out that the mileage of railways in the United States amounts to 10% more than that of all the railways of Europe, though the population is only 90 000 000, as compared with 450 000 000 in Europe, he states:

"Before the railways [of the United States] became developed, attempts were made to extend the inland waterways by constructing many canals, but inland navigation gradually died out, except on the Great Lakes, where it has a quasi-maritime character, and on some

exceptionally placed waterways, as soon as a better sytem of transport became known. It was the railway which made it possible to develop with unexampled rapidity this great continent, to develop farms whose produce, sent to the Old World at very low prices, there produced the agricultural crisis of thirty years ago. Then it helped to extend a population there which will very soon consume all the produce of its own country, so that its growth is an important factor in the general rise in prices, which is now producing with us a crisis in an inverse sense of the former one, namely: the dearness of food. Finally, the railways, which carried coal and minerals at rates even lower than those charged for cereals, helped to give the United States an industrial development, the general growth and magnitude of which are just as surprising as the former development of farming."

Statistics are then given showing in detail the much lower costs of transportation in the United States as compared with those of Europe,* and he continues:

"In Europe 10-ton wagons are still the rule, 20-ton wagons are exceptional, and 40-ton wagons are very rare. In the United States, out of 2 400 000 wagons, there are only 12 000 which take less than 18 tons, and most can take 18 to 33 tons; 634 000 can take 33 tons, and 390 000 can take 42 to 54 tons. These working conditions, which have resulted in the historical development of American farming and industries, make it possible to have very low rates and still work at a profit."

Summary.

The preceding discussion may be briefly summarized as follows:

Curvature.—Within the limits of the practical operation of railroads, a curvature of any radius which is feasible on meter-gauge is as practical on 4 ft. 8½ in., or even 5 ft. 6 in.

Gradients.—The working of trains on steep gradients is not affected by gauge, except in so far as the narrow-gauge cuts down the locomotive capacity, and is, therefore, at a great disadvantage.

Location.—In view of the foregoing, the narrow-gauge (3 ft. 0 in. to 3 ft. 6 in.) does not permit better adaptability of the alignment to the conformation of the ground.

Cost of Construction.—This is so little more for 4 ft. 8½ in. as to be more than counterbalanced by economy in operation.

Speed.—This is an important feature in railway operation which is adversely affected by narrow-gauge.

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* See also the statement on p. 581.

Stability.—The relative stability of narrow-gauge trains in motion is much less than that of trains on medium- or broad-gauge; consequently, high speeds are impractical and may be dangerous.

Track Stresses.—Much higher track stresses are produced on the narrow-gauge; there is greater impact and therefore more difficulty in maintaining track on narrow- than on broad-gauge.

Rolling Stock Cars.—The greater width of narrow-gauge cars in proportion to the gauge is considered a necessary evil rather than an advantage, and it is shown that there is probably no advantage to be gained by making the broad-gauge cars any wider than they are at present. In the matter of passenger cars, the narrow-gauge is at a decided disadvantage, both as to cost per unit of accommodation and in dead weight per passenger. In freight cars, there is little difference in cost, within the limits of size possible on the narrow-gauge, though cars of twice the capacity are available for medium- or broad-gauge. The increase in weight of modern freight trains, in order to decrease the cost of operation, has made heavier draft rigging necessary, thus tending to increase the proportion of dead to paying load to a much greater extent on small capacity cars than on the larger ones, than has been the case heretofore.

Locomotives.—The cost per unit of capacity of locomotives is about the same for each gauge up to the limit of the narrow-gauge, which limit for all types is about half of that of the 4 ft. 8½-in. or 5 ft. 6-in. There is an economy in the use of heavier types of locomotives not available for the narrow-gauge, and the use of cars of large capacity is a necessity for cheap transportation.

Train Resistance.—Train resistance is decreased by the use of larger units.

Capacity.—Figures are given tending to show that the relative capacity of narrow-gauge lines is not more than half that of medium- or broad-gauge.

Heavier Engines and Trains.—There is a general discussion showing the part played by increase of train loads in reducing costs in the United States.

Cost of Operation.—The ton-mile costs, on both Indian and Australian Railways, as well as those in the Argentine, are less on the medium- and broad-gauge than on the narrow-gauge, although the train-mile costs are often higher.

The lowest ton-mile costs on any railway in the Argentine are more than twice as high as any in the United States.

The saving in cost of operation, due to the use of larger cars and heavier trains, is considerable, and would in itself go a long way toward meeting the interest charge on the additional cost of the wider gauge.

Cost of Maintenance.—Maintenance of way probably costs more per traffic unit on narrow- than on medium- or broad-gauge on any but lines of very light traffic.

Maintenance of equipment should be less on medium- or broad-gauge by reason of the larger capacity, and, consequently, lesser number, of locomotives and vehicles, provided there is business to warrant their use.

Need of Efficient Transportation Machine.—The necessity of an efficient transportation machine to develop the country properly is pointed out, as well as the part played by the railways in developing the United States, the country most nearly comparable with Brazil and the Argentine.

THE MOST SUITABLE GAUGE.

We now come to the consideration of the most suitable gauge for the development of these countries. Such details of the topography, general conditions, traffic, etc., as are necessary for a proper understanding of the requirements are given in Appendix B.

In the Argentine, as in India and Australia—the only other countries which have large areas, large traffic, and different gauges—whatever the cause may be, however different any two lines may be, it is a fact that the greatest development has been along the lines of the broad- or medium-gauge railroads, and the big business is done by them. This is shown in Table 22.

TABLE 22.—RECEIPTS, IN GOLD DOLLARS, PER MILE OF RAILWAY PER ANNUM.

Country.	Year.	Gross Receipts.			Net Receipts.		
		Broad.	Medium.	Narrow.	Broad.	Medium.	Narrow.
Argentine....................	1911	$7 385	$3 450	$3 905	$2 340	$1 395	$1 000
India.........................	1906	9 870	....	3 910	4 845	....	2 090
* Australia, Government...	1911	6 985	8 135	3 515	2 565	3 165	1 190
Australia, Private..........	1911	2 040	3 850	2 510	1 000	3 055	1 150

*Leaving out South Australia, total receipts $26 700 for two gauges not divided; and Northern Territory, $1 875 narrow-gauge, too small to be averaged with others.

It is difficult to get close figures for Brazil, but for 1906—the latest for which official statistics are published—the three systems which contain among them the broad-gauge lines, namely, The Central of Brazil, The Paulista, and the São Paulo Railways, operate (counting both broad- and narrow-gauge) less than 15% of the total mileage of the country, and their receipts amount to nearly 65% of the total, averaging about $12 000 per mile of line.

Considering the actual experience of the other countries, therefore, and the preceding detailed discussion of the actual effect of gauge, there seems, to the writer, to be little opportunity for a difference of opinion that the 4 ft. 8½-in. gauge is the most suitable for almost all conditions, and that for new lines, even in, or perhaps more particularly in, mountainous districts, and where the amount of business to be expected is not large, the slight difference in cost is generally more than warranted by the possibilities of economy in operation. The question in the Argentine and Brazil, however, is whether the cost of changing the existing narrow-gauge lines is warranted, and, in the former country, whether the change should be to 4 ft. 8½ in. or to 5 ft. 6 in., and, by the adoption of this latter, unify the gauge of all the lines in the country.

The consideration of a change of gauge, if confined to the Argentine alone, is practically narrowed down to the question, whether it is possible to unify the gauges of all the railroads, or whether one gauge shall be eliminated, leaving only two instead of three? The following argument, as far as page 626, therefore, is confined to a consideration of the question as it affects the Argentine, and without reference to the adjoining countries.

The Best Gauge for the Argentine Alone.—The relative advantages of 4 ft. 8½ in. and 5 ft. 6 in. have been discussed at some length, and it is believed that it is shown fairly conclusively that 5 ft. 6 in. does not now (with the imposed restrictions as to loading gauge in the Argentine) offer any advantages, from a purely transportation standpoint, over 4 ft. 8½ in., nor would it be likely to, even if there were a possibility of taking full advantage of the extra width of gauge by designing rolling stock with the same proportionate overhang. It seems important to emphasize this latter statement, as it is generally supposed that the 5 ft. 6-in. gauge could be better utilized if full advantage were taken of the extra width, but this is not the case,

or, at least, only in a minor degree, and, if the 5 ft. 6-in. gauge is adopted, it will not be for any advantage it has over 4 ft. 8½-in., but solely in order to unify the gauges of all the lines in the Argentine.

It is a safe assertion that it is impracticable to work more than two different gauges, inasmuch as all lines must have access to the ocean ports, and although it is practical to work two gauges, by having a third rail, on the port railways, it is not practical to work three, and it is impractical to reserve a special section of a port for a special gauge.

The broad-gauge is undoubtedly there to stay, as it has the greatest mileage, the companies working it are by far the strongest financially in the country, and, to a large extent, are all controlled by mutually friendly interests which probably would not even consider a proposition to narrow the gauge.

The meter-gauge has the next largest mileage, nearly four times that of the medium-gauge, and it would be far more costly to convert the meter-gauge lines to medium- than *vice versa*. It seems, therefore, as if, between these two—the meter- and the medium-gauge—the medium might have to go. This would be particularly unfortunate, as it would be undoubtedly a long step backward, but there are many indications that a sentiment is rapidly growing against the existence of the three gauges.

The Government lines, all meter-gauge (except those in Patagonia which are 5 ft.-6 in.), have been built with the primary object of bringing the outlying sections of the country and the capitals of the Northern Provinces in touch with the National Capital, for both political and military reasons. They reach every provincial capital, except Entre Rios, Corrientes, Mendoza, and Buenos Aires, and are connected with the latter, the National Capital, by two separate lines of meter-gauge, the Central Cordoba and Compañia General de Buenos Aires, so that every capital (except Mendoza) and practically the whole of the northern end of the Chilian frontier and the frontier of Bolivia can be reached by through trains from the capital without change of coaches.

The frontiers of Paraguay, Brazil, and Uruguay, where they join the Argentine, can only be reached by the medium-gauge, and, though this gives access to them from the National Capital itself, if it becomes necessary to send troops to any of these frontiers, all, except those from the City of Buenos Aires and vicinity, would have to be transferred from cars of a different gauge. This, though it may appear to

be a far-fetched argument for changing the gauge of the Entre Rios and North East Argentine Railways to narrow-gauge, at least partly explains the growing sentiment that the Government should be able to reach these frontiers over lines of the same gauge which now enables it to reach every other part of the country except Bahia Blanca, and access to this latter by a meter-gauge line is only a question of time, if the rest of the meter-gauge lines remain as they are. The experience of Japan (see page 639), however, shows the inferiority of the narrow-gauge for military purposes.

There is also the feeling of the people of these two provinces, Entre Rios and Corrientes, that they are isolated. So long as their railroads stopped at the edge of the rivers, and they were obliged to leave the train and cross the water in some kind of a boat, they accepted that as inevitable. The opening up of the Entre Rios car-ferry route to Buenos Aires, however, has completely changed the aspect of the province, and has shown its inhabitants the great advantage of through railway communication, so that they will not be satisfied until they have the same kind of through communications with the great northern section of the country, Tucuman, Bolivia, and the Chaco, by car-ferry transfers between Parana and Santa Fé and between Corrientes and Resistencia.

From Santa Fé there are narrow-gauge lines reaching practically the whole country except the Cities of Bahia Blanca and Mendoza. It is extremely natural, therefore, that it should be proposed, as it is quite seriously, to cover the whole of the Provinces of Entre Rios, Corrientes, and Missiones with a network of narrow-gauge lines connecting at Parana and Santa Fé with the narrow-gauge net on the other side, or to have the Government take over the existing line and change the gauge to meter.

It seems to the writer, however, that these proposals are made without an adequate realization of what an efficient transportation machine is, and what it means to the future development of the country. The need of fairly rapid transportation for passengers, the character of the freight traffic, etc., are indicated in Appendix B.

The principal defect of the narrow-gauge, which has been already pointed out, is its much lesser stability and greater stresses, both to rolling stock and track, caused by the unevenness of the latter, combined with the much greater height in proportion to the width of support of the former. This makes stone ballast—a very expensive article in

the Argentine—an absolute necessity for the narrow-gauge, whereas the broad- and medium-gauge, in many cases, can handle the traffic fairly comfortably without it.

In considering the merits of the two gauges, there seems to be little need, at this late day, to prove that the narrow-gauge, as a transportation machine, that is, regarded solely from the standpoint of a machine for the safe, comfortable, rapid, and economical transportation of passengers and freight, is inferior to medium or broad. Even the most ardent advocates of narrow-gauge admit the inferiority, but claim that this is compensated by the lower first cost, and also, sometimes, that narrow-gauge lines are cheaper to operate.

The statement that narrow-gauge lines are cheaper to build is fallacious, but still persists in some quarters, because many narrow-gauge lines actually are built more cheaply than some medium- or broad-gauge lines. This, however, is not because they are narrow-gauge—at least not to any great extent, as has already been shown in detail—but because they are built of cheaper materials. They have lighter or lesser capacity. For the majority of the lines to be built in the Argentine, which will cost mostly less than $25 000 per km., the costs would be, approximately, for the same type of construction and including all interest and overhead charges:

Narrow-gauge, per kilometer		....	$25 000,	gold.
Medium-gauge, “	“	....	27 500	“
Broad-gauge, “	“	....	27 750	“

There will be many hundreds of kilometers costing not more than $15 000, on which the difference would be even less.

The mistake most often made in discussing this question is in confusing the terms “Narrow-gauge railway” and “Light railway,” though, of course, a “Light railway” may be of any gauge.

In some of the discussions on this phase of the subject, it has been claimed that if standard- or broad-gauge roads or branches were built with light rails, bridges, etc., it would not be possible to use the same rolling stock on them that is used on the main line, or lines of heavier construction, but this can hardly be supported. It must not be forgotten that the force of impact due to speed is a most important factor, governing the necessary strength of the track and other structures of a railroad, and though one would not expect to run the heaviest type of

locomotives over a light line, it is perfectly feasible to carry any rolling stock, except the heaviest locomotives, at low speeds on rails as light as 40 lb., and there is no economy in building bridges for any railway traffic that will not carry freight cars of 50 tons capacity running at speeds of 20 miles per hour, which is quite fast enough for branches of light traffic.

In regard to cost of operation, there is room for difference of opinion, as it seems to be impossible to offer proof of the actual effect of gauge on this. The matter has been discussed in some detail, and there seems to be no reasonable doubt that, assuming equal conditions, the cost would be less on the broad-gauge than on the narrow-, but, just how much, it would be difficult to say. Certain of the figures seem to show that the saving made possible by the use of larger rolling stock might alone be more than enough to compensate for the cost of conversion.

The problem, therefore, so far as only the Argentine is concerned, seems to be narrowed down to the question of either the unification of the gauge by conversion of all lines to 5 ft. 6 in., or of changing all the narrow-gauge lines to medium, or all the medium to narrow, and in any case the scheme adopted must embrace all the lines of the gauge to be changed. So far as having an efficient transportation machine is concerned, the problem would be solved by the conversion to the medium-gauge, but if this is to be considered seriously, the question naturally arises would it not be throwing away a great opportunity to go farther and convert all to 5 ft. 6 in., and so unify the gauge of all the lines in the Argentine (except certain of the mountain lines of light traffic which could remain as narrow-gauge feeders, at least for some time)? The additional cost of conversion to the broad-gauge instead of the medium would not be so very great, in comparison with the whole cost in a scheme so large as this, and it would not be unreasonable to assume that, to obtain a result of such far-reaching benefit to the country, the Government of the Argentine, like that of Australia, would assume a large share of the burden.

Although there can be little room to question the statement that, for new lines in a country where construction is as light as it is in the Argentine, the additional extra cost for the wider gauge is so small as not to be worth considering, when compared with its great advantages over the narrow, the important question is the justification,

from a financial standpoint, of spending the amount necessary for the conversion of the existing lines, which, up to the present at least, have not shown such great deficiency in handling their business as would seem to warrant the expenditure of this sum in improvements.

It must not be forgotten, however, that, as pointed out on page 604 and in Appendix B, the character of the narrow-gauge lines is just now undergoing a radical change, from that of more or less small local lines, doing a small local business without competition, to that of a large trunk-line system, many of the lines of which are or will be competitive either with other broad-gauge lines or with river transportation, and they must either develop into an efficient transportation machine, capable of handling such business as may be offered at rates and in a manner which will compare favorably with those elsewhere, or be eventually crowded to the wall by transportation systems which can and will do this.

The underframes and draft rigging of the larger part of the rolling stock of the narrow-gauge lines is so light as to prevent its use in trains of any weight or length. The strengthening of these, or the provision of adequate strength in new equipment, will make the dead weight relatively large in proportion to the paying load, and in every way the narrow-gauge is forever limited in the matter of speed, comfort, stability, and economy of operation when compared with the standard-gauge.

If no great development or extension of the existing system of railroads were to be expected, it is doubtful if the expenditure necessary for conversion would be justified, but if the future development of the country is to be on the scale which a knowledge of it and a study of the general condition of the world's food market seem to indicate, it is believed that it can be.

The possibilities of double-tracking the existing narrow-gauge lines instead of widening the gauge, must, of course, be considered, especially as the cost of this on the lighter lines would be little, if any, more than the cost of conversion, and double-tracking could be carried out as required and without disturbing traffic in any way. It is doubtful, however, if a double-track narrow-gauge line has much greater efficiency or even capacity than an efficiently managed single-track line of standard-gauge (12 000 000 tons per mile of line per annum, or a total of 650 000 000 tons per annum, are handled on the Bald Eagle Branch, a single-track line of the Pennsylvania Railroad, 54 miles in length),

and then the cost of maintenance of double track is practically double that of single track. (See also page 580.) The cost of maintenance per ton-mile for any volume of traffic which might reasonably be expected in any part of the Argentine, except in the vicinity of the National Capital, would probably be almost, if not quite, twice as much for a double-track narrow-gauge as for a single-track wide-gauge, and the possibly somewhat greater ease of handling trains would be more than offset by the other operating economies possible on the wider gauge. In mountainous or even rough country, of course, the cost of double-tracking would far exceed the cost of changing the gauge, and it has been shown that the narrow-gauge offers very little advantage in decreased original first cost in rough country.

Looking at the question from the broad economic standpoint of the future development of the Argentine, it seems to the writer that the policy of continuing the construction of narrow-gauge lines is wrong, especially as it is believed that it can be shown that the additional cost of an efficient transportation machine, which the narrow-gauge is not, can be fully justified from the financial point of view, as well as from the point of view of its absolute necessity for the proper economic development of the country.

Cost of Unification for Argentine Railways Alone.—In calculating the cost, one must consider, not only the cost of changing the existing lines, but looking farther ahead, estimate the additional cost of the completed system as broad-gauge, over and above what it would cost to complete it as narrow-gauge. That is to say, supposing there are 10 000 km. of meter-gauge to be changed and 10 000 km. more to be built; the actual cost of changing might be $8 000 per km., the cost of building as 4 ft. 8½ in., possibly $1 000 or $2 000 per km. more than if built as meter-gauge, so that the added cost per kilometer for the whole of the completed system would be about $5 000 per km.

Taking the country only as far north as 22° South Latitude (the southern border of Bolivia), there is, north of the line of the existing railway from Sante Fé to Tucuman, a region comprising some 100 000 sq. miles (260 000 sq. km.) in the Argentine alone, and say, roughly, 25 000 sq. miles (65 000 sq. km.) more in Paraguay west of the Paraguay River at present served by only about 1 250 km. of railway. In other parts of the Argentine the length of railways in comparison with the area (for 1909) is approximately as shown in Table 23.

TABLE 23.

	Per 100 sq. miles.
Federal District	152.0 km. = 94.0 miles.
Province of Buenos Aires	7.2 " = 4.5 "
" " Cordoba	5.0 " = 3.1 "
" " Santa Fé	8.0 " = 5.0 "
" " Entre Rios	3.1 " = 2.4 "
Similar units for other areas are:*	
United States	14.1 " = 8.75 "
France	27.8 " = 17.22 "
Prussian-Hesse	40.0 " = 24.08 "
Great Britain	52.6 " = 32.65 "

* *Bulletin*, Bureau of Railway Economics.

It seems reasonable to assume that to develop this northern region there should be at least 5 km. of railway per 100 sq. miles, or a total of 6 250 km., of which there is now in existence some 1 250 km., leaving 5 000 km. yet to be built.

There is in existence at the present time in the whole country about 9 000 km. of narrow-gauge railway, of which 1 000 km. are strictly mountain lines of light traffic which need not be converted for some little time. In the next 20 years there will be required, say 4 000 km. of extensions to the existing lines, outside of this northern zone. So that, taken altogether, mountain lines and all, there will be 9 000 km. of the narrow-gauge to be converted and, say, 9 000 km. of new lines to be built.

To complete the unification of the gauge to 5 ft. 6 in., the standard-gauge lines must be included, of which there will be about 2 500 km. to be converted and about as much more new line required for extensions. The total additional cost of the final broad-gauge system, as compared with narrow-gauge, would then be:

Conversion of 9 000 km. of meter-gauge to broad, at $10 000 per km.	$90 000 000
Extra cost of building, say 9 000 km. of broad-gauge instead of meter, at $2 500	22 500 000
Cost of changing 2 500 km. of medium-gauge to broad, at $6 500 per km.	16 250 000
Extra cost of building 2 500 km. of broad-gauge instead of medium, at $1 500	375 000
	$129 125 000

That is looking ahead, say, 20 years, and not considering the existing broad-gauge lines. The average cost per kilometer of line then in existence as broad-gauge would have cost about $5,000 per km. more than if the existing narrow- and medium-gauges are perpetuated.

The interest on this, at 6%, is approximately $300 per km. per annum, so that, taking into consideration all the figures given in the previous discussions, and the several other undeniable advantages pointed out, it seems entirely reasonable to expect that, even with only a fair amount of business, the earning capacity of the broad- over the narrow-gauge would justify this expenditure, and, in addition to this, it is believed that there are many other reasons, particularly the narrow margin of safety on the narrow-gauge, the necessity of keeping the track in almost perfect condition in order to insure the possibility of even reasonable speeds and a fair amount of comfort, which warrant the expenditure.

The total expenditure indicated in the foregoing makes full allowance for the losses, if any, which might be due to the change of rolling stock. If, as has been proposed, the change was spread over, say, 10 or 12 years, quite a large proportion of the existing rolling stock would be used up during that period, and at the end of that time, in all probability, there would be direct connection with the narrow-gauge lines of the West Coast and the remainder of the rolling stock could be disposed of to roads in those countries, thus greatly minimizing this item; also, the difference in the value of the old rolling stock and the new would be much more than counterbalanced by the economies possible in the operation of the new rolling stock, all of standard types and of similar classes, and all selected in the light of the present knowledge of the requirements of the conditions to be met and the wide range of types now available to meet them.

The Best Gauge for South America.—The foregoing argument is based entirely on the point of view of the Argentine alone, without considering any of the adjoining countries or the necessities of international communications. If the whole of that part of South America referred to, that is, the part shown on the map, Plate I, is considered, however, the solution of the question as to which of the two wider gauges to adopt, 5 ft. 6 in. or 4 ft. 8½ in., is not quite so apparent.

The adoption of the 5 ft. 6-in. gauge in the Argentine leaves Paraguay and Uruguay with the medium (4 ft. 8½ in.), and it can hardly

be expected that either of these countries, more especially the latter, with some 2 290 km. of line, would consider a change from which it could expect no possible benefit. Brazil, because of the existence of a small quantity of 5 ft. 3-in. gauge, might possibly be inclined to favor the 5 ft. 6 in., but there is not enough of it really to influence the question either way. The 5 ft. 3-in. is neither one thing nor the other, and it would be practically as easy to change it to 4 ft. 8½-in. as to 5 ft. 6-in.; therefore, it seems far more likely that, if the change could be brought about, Brazil would prefer to change to 4 ft. 8½-in. as a standard, thus conforming with Uruguay and Paraguay and the existing lines in Entre Rios and Corrientes.

For the 19 000 km. of narrow-gauge lines now existing in Brazil it would cost about $40 000 000 or $50 000 000 less to convert them to standard-gauge than to 5 ft. 6 in. Looking toward the north, also, the lines of Peru are largely standard-gauge, even where they cross the highest passes of the Andes, and this really is the only logical gauge to adopt if an agreement could be effected between the different countries.

The conversion of the meter-gauge lines of the Argentine to standard 4 ft. 8½ in., and the adoption of that gauge for all new lines north of Santa Fé and Cordoba; and in the rest of the country, except for the legitimate extensions of the existing broad-gauge lines, would be a much less costly proceeding than that estimated above for their conversion to 5 ft. 6 in. It would provide an efficient transportation machine reaching from Buenos Aires to all parts of the country, and in harmony with the network already existing in Uruguay, Entre Rios, Corrientes, and Paraguay, and it might not be unreasonable to expect that, with this in view, Brazil would join in the movement, as there can be no question that such large areas as those under consideration cannot be efficiently developed by a transportation machine inferior to that which has achieved such wonderfully successful economic results in the United States, with which country this part of the world is most nearly comparable.

Making the estimate for the Argentine alone, on the basis of a change to 4 ft. 8½ in. instead of 5 ft. 6 in., and leaving out the existing 5 ft. 6-in. system, the cost of the completed system of standard-gauge roads would be as shown in Table 24.

TABLE 24.—COST, IN GOLD DOLLARS.

Conversion of 9 000 km. of meter-gauge to 4 ft. 8½-in. at, say, $8 500 per km.	$76 500 000
Additional cost of building, say, 9 000 km. of 4 ft. 8½-in. instead of meter, at $2 000 per km.	18 000 000
Extra cost of building 2 500 km. of extensions in Entre Rios, Corrientes and Missiones	nil.
Cost of changing existing 2 500 km. in Entre Rios, Corrientes and Buenos Aires	nil.
Total	$94 500 000

That is, the standard-gauge system in existence at the end of 20 years would have cost about $4 000 per km. more than if the meter-gauge had been perpetuated. The interest on this at 6% would be about $240 per km. per annum, which it is believed could easily be saved by economy of operation, to say nothing of the necessity of having an adequate transportation machine.

Next, take the area south of 22° South Latitude and between the Atlantic Coast and a line drawn from Buenos Aires along the river to Santa Fé and passing through Tucuman and Salta to Sucre, but omitting the section north of São Paulo and Rio, already fairly well served by railways. There are in this area about 1 000 000 sq. miles, served by only about 12 200 km. of railroad or 1.22 km. for every 100 sq. miles of territory.

Of this area, the Province of Entre Rios is, perhaps, the best served at present, with about 3.9 km. of railway per 100 sq. miles, and it is seriously proposed to double this mileage within the next few years.

The railway mileage of the Argentine has increased three-fold in the 20-year period, 1890 to 1910, and it seems not unreasonable to expect the same rate of increase during the next 20 years in the area under consideration. It is true that the climatic conditions are not quite so favorable, but, on the other hand, the era of great expansion in North America, through the development of the country by new lines of railway, is now over, the world demand for pastoral and agricultural products is increasing, even the United States itself is just entering into the class of consumers, so that the development of this area seems inevitable, and will only be held back, if at all, by the lack of capital.

Assuming that there will be 25 000 km. built in this territory during the next 20 years, which will give a density of 3.7 km. per 100 sq. miles,

of this, probably, 5 000 km. will be of standard-gauge, in any event. Then the cost of a standard-gauge system throughout the whole area, including the cost of conversion of all the existing lines except those of broad-gauge of the Argentine, would be as shown in Table 25.

TABLE 25.

Cost of conversion of 28 000 km. of meter-gauge to standard, at $9 000..	$252 000 000
Additional cost of building 20 000 km. of line, 4 ft. 8½-in. instead of meter, at, say, $2 500..	50 000 000
Cost of conversion of 500 km. of 5 ft. 3-in. to 4 ft. 8½-in., at $1 000....	500 000
4 800 km. in Uruguay, Entre Rios, Corrientes, and Paraguay not requiring change..	nil.
5 000 km. of new line in Uruguay, etc., etc............................	nil.
Total additional cost.................	$302 500 000

It will be noted that somewhat larger allowances for conversion, etc., are made for this whole system than for the Argentine alone.

The standard-gauge system in existence over this whole area at the end of 20 years, therefore, would cost about $5 000 per km. more than if the meter-gauge were perpetuated, this involving an annual interest charge of, say, $300 per km. The greater cost by including the lines of Brazil is due to the much greater present development of the meter-gauge system there, and also by additional allowances to cover the additional costs, as a great deal of the topography is much more accidented in Brazil than in the Argentine. On the other hand, no account has been taken of the vast areas in Brazil north of Latitude 20°, in much of which, especially in Eastern Brazil, there is likely to be considerable development by new lines, which, if built in the first place as standard-gauge, would reduce the average cost per kilometer of the whole system.

It seems to be not unreasonable to assume that, as many of the narrow-gauge lines will require stone ballast at a cost of from $1 500 to $2 500 per km., where the wider gauge could be operated just as well without, the completed system should be given some credit on this account, which might easily be as much as $1 000 per km. over the whole system, thus reducing the interest charge by $60 per km.

There is also the question of double-tracking many of the narrow-gauge lines, which would not be necessary if they were medium- or broad-gauge, and this would offset quite a little of the cost of conversion, though it is somewhat difficult to say just how much. Suppose, how-

ever, that 15% of the systems as narrow-gauge would have to be double-tracked and half of this, say 4 500 km., could be avoided by the change of gauge, there would be a saving of about $100 000 000 on this item alone, or one-third of the estimated additional cost of the whole system as standard-gauge.

It has been shown that, on the basis of the most conservative estimates, the saving in cost of operation made possible in only the one item of larger rolling stock, might easily be more than $300 per km. per annum, and as it is considered necessary even now to use 70- and 80-lb. rails on the narrow-gauge, this is not offset by the necessity of increasing the strength of the track to take care of the heavier rolling stock, as this weight of rail is ample to carry the ordinary standard-gauge equipment of fairly heavy types, on which these estimates are based.

It seems almost superfluous to point out the advantages of unification of the gauge. Besides the inconvenience of transfer and the actual cost of moving freight from one car to another, which has been variously estimated at from 5 to 10 cents per ton in India,* there is the actual damage to many classes of freight, such as fruit, etc. The transhipment of refrigerated products is impractical. Two cars are standing idle while any transhipment is being made, there are the unavoidable delays in yards waiting for the necessary cars of both gauges to be provided and shifted into position, and, above all, the loss in fluidity of rolling stock, which would be especially noticeable in both the Argentine and Brazil, where, for a few months during and after the harvest, there is an exceptional demand for rolling stock which could be used during the other seasons for hauling timber.

In the Argentine this question of transfer from one gauge to another has not become acute up to the present time, as most of the lines of each of the gauges reach terminal points on the water-front, to and from which most of their traffic flows. With the development of the country, the actual interlocking of the lines of the different gauges, and the present tendency to continue this mixing of the territories, this question, however, is bound to assume greater and greater importance, and the necessary transfers will be the cause of more and more inconvenience and delay.

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* *Minutes of Proceedings*, Inst. C. E., "The Railway Gauges of India," by Sir Frederick Robert Upcott, Vol. CLXIV, p. 196.

It is true that the foregoing figures are based largely on assumptions, and may be varied widely in many individual cases, but it is believed by the writer that on the whole they represent a fair average, and at least tend to indicate that the adoption of 4 ft. 8½ in. as a standard, and the conversion of the existing narrow-gauge lines, can be justified on economic grounds and approximately so from the financial viewpoint.

It is hardly to be expected that these figures are definite enough to justify private capital in entering on an enterprise of such magnitude, but, taking into consideration the fact, which it is believed has been demonstrated, that the narrow-gauge is absolutely inadequate for the development of such large areas as those under consideration in the Argentine, Eastern Bolivia, Brazil, Paraguay, and Uruguay, to say nothing of the rest of South and Central America, it may not be unreasonable to expect such a measure of assistance from the Governments involved, which would be justified by the benefits they would derive from a system of railways of uniform gauge covering the whole area.

This argument, it will be seen, is based largely on the supposition of the continued and continuous development of these countries, which seems assured, provided the past policy of encouragement and appreciation of private capital is continued and the political situation remains as stable as it now seems to be. Without foreign capital, of course, this development will be tremendously retarded, to the benefit of competing countries like South Africa, Australia, and New Zealand.

It has been, and still is, the policy of the Governments of both Brazil and Argentine to build as State enterprises lines of railway to connect up or push the development of certain sections not sufficiently attractive at the time in themselves to private enterprise. Of course, the extent to which this can be carried out is more or less limited, as in all these newer countries there is a demand for more capital than is available; therefore, the Government lines, after a while, are sold or leased, and the capital thus acquired is used for further development elsewhere. It seems to the writer that, taking everything into consideration, such lines, if built to standard-gauge, would cost little if any more than have the meter-gauge lines, and there is no question at all but that they would be a far more valuable asset because of their undoubted greater economy in operation and their better salability

because of the wider market for any standard article than for one of inferior quality.

The 5 ft. 6-in. lines of the Argentine probably will not consider the change for the present, but, as they are fairly well confined to a limited area, it will do no harm to leave them as they are. They are efficient, and to attempt to include them in any scheme for unification of gauge would be to create a formidable opposition which it would be difficult to overcome. The adoption of 4 ft. 8½ in. as a standard for new lines, and the gradual change of the meter- and other narrow-gauge lines to the standard, however, should be established as the policy, if the requirements of the future are to be met.

It must be borne in mind that, as the distance from the seaboard increases, the necessity for cheap transportation becomes more and more important, and as this section of the world must depend for its future prosperity on its ability to compete successfully in the food markets of the world, cheap efficient transportation is a *sine qua non.* The United States has achieved results in this line which are far ahead of those obtained elsewhere, almost wholly by reason of the increase in train loading. It is quite certain that no narrow-gauge line can attain anywhere near the economic results obtained in the United States, and nothing which falls much short of this will solve the problem for the area under discussion, which only needs capital and adequate railroad transportation to show the most phenomenal development in the history of the world. This development, however, will be tremendously handicapped with anything less than the efficient, economical transport which can be furnished by standard-gauge railways.

In closing, the writer wishes to express his thanks to Mr. Percival Farquhar, President of the Argentine Railway Company, for permission to use much of the information contained in a report made to that company on this subject by the writer, to Mr. L. E. Young, of the Middletown Car Company, for data in regard to rolling stock, and to Mr. C. M. Muchnic, of the American Locomotive Company, for data in regard to locomotives and other information. It is believed that proper credit has been given in the text for quotations and assistance from engineering periodicals and other sources, but inasmuch as a fairly wide search of the literature of the subject has been made,

it is possible that some omissions of credit may have been made, if so, it is needless to say it has been unintentional.

Generally speaking, it is believed that the statistics given in the paper are substantially correct, and in any event such differences as there are do not affect the general argument, so far as the gauge question is concerned. As with all figures relating to railroads, there are often differences due to different methods of calculation or recording, and this is perhaps especially so in regard to South America. The writer, therefore, would esteem it a favor if those who may have more intimate knowledge of any facts will not hesitate to state them with a view to making the record as complete and accurate to date as may be.

APPENDIX A.

History, Experiences of Other Countries, Etc.

The following notes give briefly some account of the unification of the gauge of the railways in the United States and Mexico, the experience with lines of different gauges in India, Australia, etc., and Table 26 shows the length of lines of various gauges now in operation in the different countries of the world:*

TABLE 26.—Length of Railways of Various Gauges in Operation Throughout the World.

All Distances in Kilometers.

	Broad, more than 5 ft.	Medium, 4 ft. to 4 ft. 11 in.	Narrow, 3 ft. to 3 ft. 11 in.	Industrial. less than 3 ft.
Great Britain	4 959	40 017	909	104
Europe (Continent)	71 142	206 912	23 517	8 264
North America	....	447 721	12 394	....
Central America and West Indies	153	3 382	2 596	141
South America	18 852	6 663	37 115	3 094
Asia	43 781	11 650	41 299	2 618
Australasia	6 415	6 179	19 257	226
Africa	....	5 275	27 843	4 039
Totals	145 302	727 799	164 930	18 486

Note.—The medium-gauge is nearly all 4 ft. 8½ in.
The narrow-gauge is nearly all meter or 3 ft. 6 in.

United States.—The following, prepared by Mr. George L. Fowler, of Messrs. Hildreth and Company, gives a brief statement of the history of the gauge discussion and its result in the United States.

"The narrow-gauge mania which possessed the railroad builders of the United States in the early Seventies was born of the bargain hunting desire to get something far below its real value. It was at the time very difficult if not impossible to raise money for the construction of new railroads of the standard-gauge. It was just after the civil war, and hard times were upon the country, yet in spite of this fact, the people over the whole land were clamoring for a more rapid development of the resources than was taking place, and this clamor was especially loud in its demand for an increase of railroad construction, but capital could not be obtained.

"It then occurred, to those who were most vitally interested in these constructions, that if cheaper roads could be built, the money for their construction might be obtained. They immediately, therefore, jumped to the conclusion that, because a gauge of 36 in. is only about three-fifths of 56 in., a road of that gauge could be built for a correspondingly lower price, and straightaway positive statements were made to that

* Compiled from "Universal Directory of Railway Officials," 1912.

effect, which were honestly accepted by a large number of men. It was assumed that, because the gauge was to be narrowed, all costs would be cut down in a corresponding degree. It was argued that because of the narrow-gauge, the cars would be made lighter than those which it had been found advisable to use on the broad-gauge, forgetful of the fact that equipment of the same, or even lesser, weight could and had been built and used on the standard-gauge lines, and had been found unsatisfactory.

"The whole case rested, apparently, so far as the United States was concerned, on the inability to raise money for ordinary railroad construction, and the narrow-gauge seemed to promise greater dividends on a given investment. It was characteristic of the whole discussion that the advocates of the narrow-gauge insisted throughout that their construction was the cheaper, and could not or would not see their opponents' side of the case, which was, that the narrow-gauge could not be built materially, if any, cheaper if the same facilities and capacity for the transportation of freight and passengers were to be afforded. They failed to see or acknowledge that a light locomotive and car could be built for a broad-gauge line.

"The physical disadvantages of the narrow-gauge, such as cramped storage and seating capacity, the greater instability of the rolling stock. and, above all, the chief drawback of the impossibility of interchanging cars with the standard lines, with the consequent extra cost and delays in transportation, were pointed out again and again, but in spite of all this the narrow-gauge construction went on for several years, and, until the money market eased, made a great headway.

"Meanwhile, the disadvantages of different gauges were making themselves manifest, and it was also being realized, with great rapidity, that heavy loads meant lesser costs than when light ones were hauled, so that between 1873 and 1885 there was a very rapid increase in the carrying capacity of the standard-gauge cars, rising as it did from 10 to 30 tons. With this rise the dead weight of freight cars dropped from a ratio of 1 lb. of dead weight to 1 lb. of paying load to ½ lb. of dead weight to 1 lb. of paying load.

"When the necessity of car interchange drove the broad-gauge lines to change to the standard (4 ft. 8½-in.) gauge, the narrow-gauge lines were left in a state of greater isolation than they had been at the outset of the movement.

"Finally, the recuperation after the panic of the Seventies made money more easily procurable for railroad construction, and the narrow-gauge roads, which had been built and equipped in the lightest and cheapest possible manner, found themselves doubly handicapped by their lack of facilities to do business, and by their isolation.

"The result of all of it was that, the furore past and the money market relieved, the narrow-gauge mania died a natural death, if a death from lack of substance can be called natural. The result then, of the forty years of practical experience that has elapsed since the battle of the gauges was precipitated in the early Seventies, has been to quite discredit the narrow-gauge system. It has been proven beyond all peradventure that the narrow-gauge railway cannot be operated so as to keep its ton-mile costs down to the figure obtained on the standard-gauge roads, and this has been the cause of its demise.

"Cars and locomotives of present capacities are impossible on a 3-ft. or 3 ft. 6-in. gauge, and time has shown that the contentions of the early opponents of the system were correct and that the narrow-gauge railroad cannot compete with the broad-gauge in capacity, facilities offered, or in cost of operation, when this cost is put upon the ton-mile basis."

The Denver and Rio Grande was the most important of the narrow-gauge lines changed to standard, and is interesting from the fact that it is a mountain line, of steep grades, very sharp curvature, and heavy work, crossing as it does the mountainous region of Colorado from Denver to Salt Lake City.

The change was started in 1886, during which year almost half the mileage (1 200 miles) was changed, the rest being done as the finances of the road permitted, and less than 6 months ago (September, 1912) it was decided to complete the last section, over Marshall Pass, a distance of 236 miles, the cost of which work, with the improvements in the line, is estimated to be about $2 000 000, or about $8 500 per mile.

It is of interest to note that on this line, which is now a link in a transcontinental route, and reaches an elevation of more than 10 000 ft., there are grades of 3.8% and curves of 150 ft. radius operated by heavy locomotives, and on which passenger trains of standard Pullman equipment are operated.

Many other lines in the United States were changed, but the largest mileage was in changing from 4 ft. 10-in., 4 ft. 11-in, and 5 ft. 0-in. to the standard 4 ft. 8½-in., this involving some 25 000 miles of various lines, mostly in the South. The total mileage of narrow-gauge changed to standard was probably about 5 000.

Mexico.—The principal railways in Mexico were built to standard gauge, 4 ft. 8½ in.; the gauge of the Mexican National, however—the original trunk line from the frontier of the United States to the City of Mexico, 1 200 miles in length—was made 3 ft. 0 in. Notwithstanding that it had a much shorter direct line to the United States, it was unable to compete with the Mexican Central and the International. In his Annual Report in 1889 the President of the Company said:

"Having been brought by experience to a realization of the inadequacy of a narrow-gauge road to develop a thoroughly satisfactory transportation service for a large volume of business * * * induced your management to take up some time ago the study of the practicability and desirability of changing the track to standard-gauge."

The line was changed to standard gauge between 1902 and 1904.

India.—The gauge of the Indian Railways has been the subject of almost endless discussion. Although nearly all who are familiar with the subject agree that a difference in the gauge of continental railways, or in countries as large as India, is unfortunate, all the discussion and the various reports of eminent engineers and transporta-

tion experts have led nowhere, and lines of both gauges are continuing to be built, apparently, however, only because the problem of conversion seems to be too big to handle. A few quotations from some of the discussions will show the attitude of some of the most prominent British engineers who have studied the subject. The matter was brought up officially for discussion by the Institution of Civil Engineers in 1906, the paper and discussion at that time occupying 135 pages of the *Minutes of Proceedings,* and, previous to that, at intervals as far back as 1873,* the following, at that time far-sighted, opinion was expressed:

"While the 3¼-ft. gauge might answer for the carriage of heavy minerals in special districts, the general commerce of every populous country mainly consists of articles of low or medium specific gravity, such as food, clothing, fuel, etc., averaging about 80 cu. ft. per ton weight, for which the 5½-ft. gauge is in every respect most suitable as regards costs, stowage, safety, economy, the intricate elements of military defense, and the power of adopting single-track lines of railways for the accommodation of a large amount of traffic."

Mr. Thomas Robertson was appointed in 1903 as Special Commissioner by the British Government to report on the administration and working of Indian Railways, and in his report he pointed out the difficulties of working the two gauges, meter and broad, as follows:

"It will generally be admitted that a break of gauge is a drawback always, and that in certain eventualities it might prove extremely inconvenient. It necessitates a great deal of expense in the transshipment of traffic and a very much larger supply of rolling stock, since the unavoidable delays during the progress of transshipment absorb a large quantity of rolling stock of both gauges.

"The question is so full of difficulty and the evil is so far advanced that it is not easy to advise on it.

"Uniformity of gauge is bound to be demanded some day, and the longer a settlement of the policy to be pursued is deferred the greater will be the difficulty and expense of introducing it."

He discusses the relative merits of the two gauges, and favors the meter-gauge, largely because of the greater width of rolling stock in proportion to the width of the gauge. (This additional width, as is shown elsewhere, is not an unmixed blessing, as it is gained at the expense of safety and stability.) He concludes by advocating the unification of the gauge by the adoption of the medium-gauge, 4 ft. 8½-in.

In 1906 a paper was presented before the Institution of Civil Engineers of Great Britain by Sir Frederick Robert Upcott, Chairman of the Indian Railway Board, who pointed out the facts in regard to the gauge of Indian Railways without much comment. They were stated to be:

* And in *The Railroad Gazette*, November, 1872.

That the internal communications of the country were more important than any considerations of communication with other countries;

That the bulk of imports and exports was by sea;

Large internal trade;

The attempt to confine the narrow-gauge to a particular zone was useless, as these lines must find an outlet to the coast, and this created a great confusion at the ports by reason of difference in gauge;

The trouble and inconvenience of transshipment were considerable, and would by no means be measured by the cost of actual transfer of articles, but involved rolling stock standing idle, lack of fluidity of rolling stock, etc., etc.

Some of the discussions were, briefly, as follows:

Sir Guilford Molesworth had nothing to say against the meter-gauge as a gauge, but he did think the standard-gauge (5 ft. 6-in.) was better suited to the bulky agricultural traffic of India.

Sir George Bruce stated that when the discussion of this question came up in 1873 he estimated that the difference in cost of construction was very large, being based on the misconception that the width of gauge was the element which regulated the cost. It did nothing of the kind. With a coach of a given width, nothing was gained by putting the wheels closer together.

Mr. R. W. Egerton, commenting on a suggestion to change broad-gauge to narrow-, said:

"It would be economically unsound to substitute an inferior means of transportation for a superior one, at the same time increasing the capital cost * * *. The meter gauge, on account of the low limit of speed attainable, was an unsuitable gauge for large trunk-lines."

Mr. F. E. Robertson observed that most discussions on the gauge question had been vitiated by the implicit assumption that all the dimensions were geometrically similar, and, also, by the omission to remember that speed was a commodity which was sometimes worth money.

The difference in running speeds between the two gauges was not really known, for the broad-gauge (in India) was in this respect usually worked in such a leisurely way, that the meter-gauge had no difficulty in keeping fairly close to it.

Sir Frederick Robert Upcott, replying to a statement that high speed was not required in India, stated:

"Mr. * * * was incorrect in his views as to the need of rapid travelling in India. India, * * * was like every other country, and had progressive ideas of speed, comfort and safety, and the Railway Administrations would have to meet these needs for all classes."

Africa.—In connection with this same discussion of the gauge of the Indian Railways, there were the following expressions of opinion in regard to the gauge of the African Railways:

The General Manager of the Cape Railways wrote (1906):

"There has been considerable agitation on the subject of the laying of light railways during the last few years. I see no reason for changing the opinions I expressed in 1891 and 1892 on this subject, and I trust that Parliament will gravely consider the probable effect of the introduction of a narrower gauge than that of our own railways before authorizing such a breach of gauge. If the gauge originally adopted, 4′-8½″, had been continued instead of changing to 3′-6″ the journey from Capetown to Johannesburg would be performed in about half the present time. The effect of such saving of time in the passenger traffic would have been enormous, but if the present gauge is to be still further reduced, what will posterity think of the foresight, or want of foresight, in adopting a standard that limits the speed and carrying capacity of the trains? When the traffic is not expected to be heavy I see no reason why railways should not be constructed with less ballast than a standard line. When the traffic improves, the line could be better ballasted and prepared for quicker speeds and heavier loads."

C. O. Burge. "The line from the Zambesi river through to Cairo had been projected on a *miserable 3′-6″ gauge*. It would be found that the steamship companies, who were progressing faster than the railway projectors, would carry passengers from North to South Africa much more quickly by steamer than the railway could on a line of 3′-6″."

Japan.—Japan, after a most careful examination of the question, has decided to change the gauge of its railways to 4 ft. 8½ in. The 3 ft. 6-in. lines of that country were found to be entirely inadequate for the transport of troops and war material during the Russo-Japanese war, and, after a most careful consideration of the whole question, it has now been decided to make the change, the estimated cost of which is said to be about $150 000 000 for the 5 000 miles of line.

The South Manchurian Railway was originally 5 ft. 0-in. gauge and during the Russo-Japanese war was converted by the Japanese to 3 ft. 6 in., this being chosen because only rolling stock of that gauge was available. After operating it as a narrow-gauge road for 2 years, however, it was changed to 4 ft. 8½ in., corresponding with the gauge of the Korean Railways, and it has been stated that the Japanese have felt that they owed a debt of gratitude to the American engineers who built the first Korean railway and thus established a gauge which was of great service to them during the war, the 3 ft. 6-in. gauge which they were obliged to adopt temporarily for the South Manchurian Railway having been found entirely inadequate.

Australia.—The situation in Australia is described in the following quotation:*

* *Engineering Record*, September 28th, 1912.

"By some unfortunate misunderstanding, each now laying the blame on the other, New South Wales and Victoria started railways in the Fifties on different systems, the first on the standard, or 4 ft. $8\frac{1}{2}$-in., and the latter on the 5 ft. 3-in. gauge, and, notwithstanding the advice of the engineers of the day, persisted, until now we have two great groups of over 3 500 miles each * * *. The slight difference of 6 in. stands much in the way of even the very inadequate remedy of a mixed gauge, owing to the difficulty and expenses connected with the construction and working of the switches and frogs in such a case.

"Subsequently, Queensland and Western Australia, attracted by the expected but exaggerated economies in the construction and working of narrow-gauge lines, which were too hastily assumed to be synonymous with light lines, adopted 3 ft. 6 in., and South Australia, after most of its main lines had been made to correspond with the width [5 ft. 6 in.] of those of its neighbor Victoria, introduced a local diversity by making its extensions and branches on the smaller, 3 ft. 6-in., basis.

"Unlike the United States and Canada who repented comparatively early in this matter, but at a cost of a large amount of money, Australia has done nothing but talk about it, and, owing to the enormous extension of each gauge in recent years, it is quite out of the question now to think of universal conversion, which might cost anything up to $60 000 000."

Finally, after the federation, it was decided to build a transcontinental line to unite the extreme easterly and westerly States of the Commonwealth, and, of course, the question of the most suitable gauge came to the front at once. After long debate and after calling to its aid the most expert advice, both engineering and other, that it was possible to obtain, the 4 ft. $8\frac{1}{2}$ in. was adopted.

There still remained in abeyance the question of the gauges of the existing lines, but it has just recently been announced* that it has been decided to unify the gauges of all the lines, adopting 4 ft. $8\frac{1}{2}$ in. as a standard and, in view of the general benefit of this to the whole country, the greater part of the burden of this change, estimated to cost about $175 000 000, will be borne by the Commonwealth as a whole.

As an example of the misleading statements often made in discussing the relative value of the gauge, the following is worthy of notice because it is so recent, and made by what one would suppose might be a competent authority.

In December, 1912, speaking at the Royal Colonial Institute in London, Sir Thomas Robinson, Agent General of Queensland, in describing the 3 ft. 6-in. railways of that part of Australia, made the statement that "Queensland had constructed 4 266 miles of railways of this gauge at practically half the cost of 3 807 miles of 4 ft. $8\frac{1}{2}$-in. gauge in New South Wales." He omitted, however, to mention the fact that the total net earnings of the latter were more than double

* *Railway Age Gazette*, July 11th, 1913.

those of the former. He also stated that "in the working of the railway the comparatively light weight of the rolling stock gave it a great economic advantage over railways constructed on the wider gauge," yet said later, "Coming to rolling stock, the lighter engines and cars of the early days ran on lighter sections of rails, but the increased weight and power of the engines had necessitated the relaying of the permanent way with far heavier rails, * * * and the use of this heavier rail was steadily and surely extending from one end of the State to the other." He claimed that break of gauge has few disadvantages, yet stated that a new route has been laid out to the border of New South Wales on which "all the stations, tunnels, etc., had been built sufficiently wide so as to run the New South Wales trains (4 ft. 8½ in.) right through".

The consensus of all these opinions, from a wide range of territory and a very large number of prominent men, covering practically the whole of the railway systems in the English-speaking parts of the world, is decidedly against the adoption of lines of different gauges in the same country, and almost entirely in favor of the broad (4 ft. 8½ in. or 5 ft. 6 in.) as compared with the narrow (3 ft. to 3 ft. 6 in.).

APPENDIX B.

General Description of the Topography, Physical Characteristics and Business Conditions of Southern South America; The Development and Present Situation of its Railways.

In the following, the Argentine is referred to perhaps more particularly than any of the other countries, but this only because the writer is more intimately familiar with it, and the data he has available relate principally to the railways of that country. The conditions there, however, are quite generally similar to those elsewhere in this south temperate zone, and may be assumed to be applicable throughout most of this area and to its railways.

The northern limit of this section for convenience may be assumed to be about Lat. 20° S., that is to say, approximately equal to that of the southern coast of Cuba and the City of Mexico; its southern limit will be taken as the City of Bahia Blanca, Lat 39° S., which is equivalent to that of Baltimore. The latitudes of some of the more important cities may be compared with some of those of the northern hemisphere, as follows:

Rio de Janeiro,	Havana.
Santos and São Paulo,	Key West.
Antofogasta,	Key West.
Asuncion,	Cape Sable, Florida.
Valparaiso and Santiago, Chile,	Atlanta, Ga.
Buenos Aires and Montevideo,	Memphis, Los Angeles.
Bahia Blanca,	Baltimore, Denver.

It should not be inferred from this that there is no development in that part of the Argentine south of Bahia Blanca and Neuquen, and including Patogonia.

The railway development in this section, however, is not large, and is most likely to be dominated, as far as gauge is concerned, by that of the four important lines in the Province of Buenos Aires, which are 5 ft. 6 in.

The principal topographical features are the main range of the Andes and the River Parana, with its tributary the Paraguay, and the River Uruguay. The Andes occupy a comparatively narrow strip along the Pacific Coast, some 150 to 200 miles wide, broadening out at the northern end of Chile and in Bolivia. Between this range and the Parana and Paraguay Rivers is the vast almost level plain reaching from considerably south of Bahia Blanca to well up into Bolivia, which comprises nearly the whole of the Argentine. Uruguay, Paraguay, the two western provinces of the Argentine, Entre Rios and Corrientes, and the territory of Missiones, are considerably rolling to hilly. In Brazil the land rises abruptly from the coast line to elevations of 2 000

ft. and more, and then slopes generally toward the Rivers Uruguay and Paraguay, the intervening country being mostly rolling and in some parts rather broken and accidented.

In the Argentine there are few, if any, rivers of importance between the Parana and the Andes, the drainage from the easterly slopes of the latter, which is considerable, disappearing in the sandy soil before it has traversed the plain for any considerable distance. Toward the north, above the confluence of the Paraguay and Alto Parana, there are two important tributaries, the Bermejo and the Pilcomayo, the latter forming the northern boundary of the Argentine and dividing it from Paraguay and Bolivia. East of the Parana and Paraguay the country is generally well watered and the bridging of the rivers is an important item.

Lest the foregoing should convey the idea that much of the area of the Argentine is arid, it may be well to note further the peculiar geographical formation of this plain. The top covering of soil, which is of the richest character, and similar in appearance to that of the so-called "black waxey" belt of Northern Texas, at the Parana River and for some distance back from it, changes to a more friable and at times almost sandy loam farther back, and is underlaid by the so-called Pampean mud or "Tosca", a hard, indurated clay, almost shale. The water from the mountain finds its way between the top soil layer and the "Tosca" and between the successive layers of this latter, and is found almost everywhere by comparatively shallow borings. Nearly all of this is good or at least fair drinking water, but some is alkaline or slightly saline, and, therefore, bad for locomotive uses.

The railroad development of the Argentine, therefore, has gone ahead under the most favorable conditions, as far as facility of construction is concerned, and there has been practically no excuse at all for the adoption of the narrow-gauge. Some of the Government lines, notably that running from Tucuman north to La Quiaca, on the border of Bolivia, are in a mountainous country, but the proportion of the total is very small, and may be imagined when it is stated that more than 90% of the alignment is tangent.

Perhaps the most important physical characteristics which affect railway construction and operation are the scarcity of stone or gravel and the high cost of fuel. The coal that is used has to be imported from Europe. The scarcity of stone makes the question of ballast important, as the broad- and medium-gauge lines stand up much better with only earth ballast than do the narrow-gauge lines, and in comparing the quality of the track, the experience of the writer with the railways of the Argentine, leads him to think that it is almost fair to say that a fair stone-ballasted narrow-gauge track is no better than a good earth-ballasted wide-gauge.

The Argentine, as is perhaps well known, derives its wealth from

its pastoral and agricultural products. There are no manufactures of importance, as there is an almost entire lack of minerals and fuel, except wood. That part of the Argentine which, thus far, is developed most, as may be seen by the density of the railway lines, is in the Province of Buenos Aires and the southern parts of the Provinces of Cordoba and Santa Fé. This is an almost treeless plain, formerly devoted to little else than cattle raising, but now gradually being converted to agricultural uses, and the raising of the finer grades of cattle and horses, alfalfa replacing to a large extent the original native grasses. Wheat is cultivated mostly in the southern part of the Province of Buenos Aires and in the vicinity of Bahia Blanca, and exported largely from that port. Maize and linseed are the important crops farther north and in Entre Rios. The principal refrigerating and packing plants are on the Rio de la Plata and the River Parana between La Plata and Rosario. Quebracho comes from north of the City of Santa Fé, and nearly all the exports of it are from that port.

With the great valorization of the lands in this section, due to its agricultural development, the cattle industry is being pushed toward the north, and there seems to be little doubt that the world will soon have to look to this section for much of its meat supply, for the production of which there are vast areas of grass lands available in that part of the Argentine north of Santa Fé and reaching up into Bolivia and Paraguay, in many parts of this latter country, in Uruguay, and in Southern Brazil.

Up to the present time, nearly all the structural timber used in the Argentine has been brought from Europe and the United States, but it is now known that there are large forests of pine and cedar in the States of Parana and Sta. Catharina, in Brazil and in the Territory of Missiones. In Paraguay there are forests of important hardwoods which will now be made commercially available, as there has just been opened up through railway communication between Asuncion, through Corrientes and Entre Rios, to Buenos Aires, and a new line is under construction from Villa Rica on the Paraguay Central toward the east which will probably open up important timber industries and reach the forests where much of the yerba maté grows.

At Tucuman, and farther north between Salta and Embarcacion, there are important sugar industries, and the business of raising early fruits and vegetables for the markets of Buenos Aires, Rosario, Montevideo, etc., is already assuming considerable proportions. Mendoza and San Juan are the centers of quite important grape and wine industries, producing annually some 3 000 000 hectoliters (66 000 000 gal.) of wine, in addition to the grapes which are shipped in very large quantities during the season all over the country and even to Europe and the United States.

The great and important product of the so-called Gran Chaco, which

embraces in the Argentine practically all that territory in the triangle with Santa Fé as its apex and the Pilcomayo River as its base, is quebracho. These trees are not unlike scrub oak in appearance and are generally too twisted and crooked to be of use for structural timber. The wood is very hard and durable, and is used for railroad ties, fence posts, and telegraph poles, the trimmings, roots, etc., for firewood, and the smaller branches for charcoal. It is very rich in tannin, and the whole logs and the extract are shipped in large quantities to Germany for tanning purposes. The whole area of the triangle referred to, so far as known—and quite a little of it has not yet been explored—comprises large areas of forests with much quebracho interspersing the open spaces where the rich grasses provide excellent grazing for cattle. There are some parts which are alkaline, some swampy, some waterless, but, on the whole, it is believed that the greater part of it is useful especially as a cattle country and where maize, for feeding on the spot if not for export, linseed, cotton, tobacco, sugar cane, etc., can be grown. The quebracho forms the bulk of the freight for the Santa Fé Railway and the southerly end of the Government lines, and this is a product which demands rolling stock of large capacity for its transportation. It is of considerable importance, as it provides paying freight for new lines from the beginning, while the country is being developed pastorally and agriculturally.

Buenos Aires is by far the most important city in the Argentine. It is both the National Capital and the commercial center, its population, 1 300 000, is one-fifth of the total of the country; it receives 80% of the imports, and the exports passing through it are 60% of the total.

The principal other ports are Rosario, the second most important city, with a population of about 100 000, La Plata, the capital of the Province of Buenos Aires, Bahia Blanca, and Santa Fé. The imports which come to these places are mostly coal and lumber, but cereals are exported from all of them as well as from quite a number of smaller places along the rivers where ocean-going steamers can lie alongside the bank and load, being served by short spurs of the railroads tapping the districts adjoining. The Port of Buenos Aires is served by a Port Railway of 5 ft. 6-in. gauge operated by the Government, with a strong feeling against admitting any other gauge. The Port of Bahia Blanca has been developed by the railroads, is served by three broad-gauge railways over their own lines, and has had a very rapid development. The Ports at La Plata, Rosario, and Santa Fé, have mixed gauge (5 ft. 6 in. and meter), the first and last operated by the Government, and Rosario by a French Corporation under a concession for its exploitation.

Most of the merchandise is imported through Buenos Aires and distributed from that city throughout the country, the imports passing through this port being about 6 500 000 tons for 1910, and the exports about 2 000 000 tons. A small proportion of the imports, about

25% of the total tonnage, and consisting principally of lumber and coal, comes directly to Rosario, and in much lesser quantities to Bahia Blanca, La Plata, and Santa Fé, with a scattering to some of the smaller ports.

The cereals are exported principally from Rosario and Buenos Aires in about equal quantities, with Bahia Blanca the next in importance, and then a scattering from a considerable number of smaller ports along the Parana and a few on the Uruguay. The cereals flow naturally to the nearest seaport, ocean-going steamers reaching as far as the City of Santa Fé on the River Parana and Concepcion del Uruguay on the Uruguay River.

Cattle (live) are shipped from all over the country (see Fig. 5) for distribution to the packing plants on the river between La Plata and Rosario. The cattle from Corrientes and parts of Entre Rios go to the plants along the Uruguay River which take, principally, the inferior grades for extracts, dried beef, etc. There is also a considerable movement of cattle from one section to another for growing, fattening, etc., and for shipment to the west coast by driving across the mountains in summer.

There is good passenger business on nearly all lines, sleeping and dining cars being in general use.

The conspicuous feature of the freight traffic, of course, is the cereals. There is a heavy demand for cars during the shipping season, February to June, and in this period quite a large proportion of cars are returned empty, this unbalancing of the traffic being noticeable on nearly all the lines.

In Brazil, of course, the most important products are coffee and rubber, the former coming from that section of the country to the north of São Paulo and Rio. This territory, as will be seen by Plate I, is served by a fairly dense network of railways which, however, are of two different gauges.

The following table shows the quantities of the principal items of export for 1906. The cotton, sugar, cocoa, and tobacco, are mostly from Pernambuco and Bahia to the northeast of Rio; rubber, of course, comes from the Amazon, and the other products from the States to the south of Rio. The quantities are long tons.

Sugar	85 000	tons.
Cotton and cotton seed	35 000	"
Rubber	62 200	"
Cocoa	25 000	"
Tobacco	23 600	"
Coffee	840 000	"
Maté	57 800	"
Flour and bran	31 500	"
Hides	27 500	"

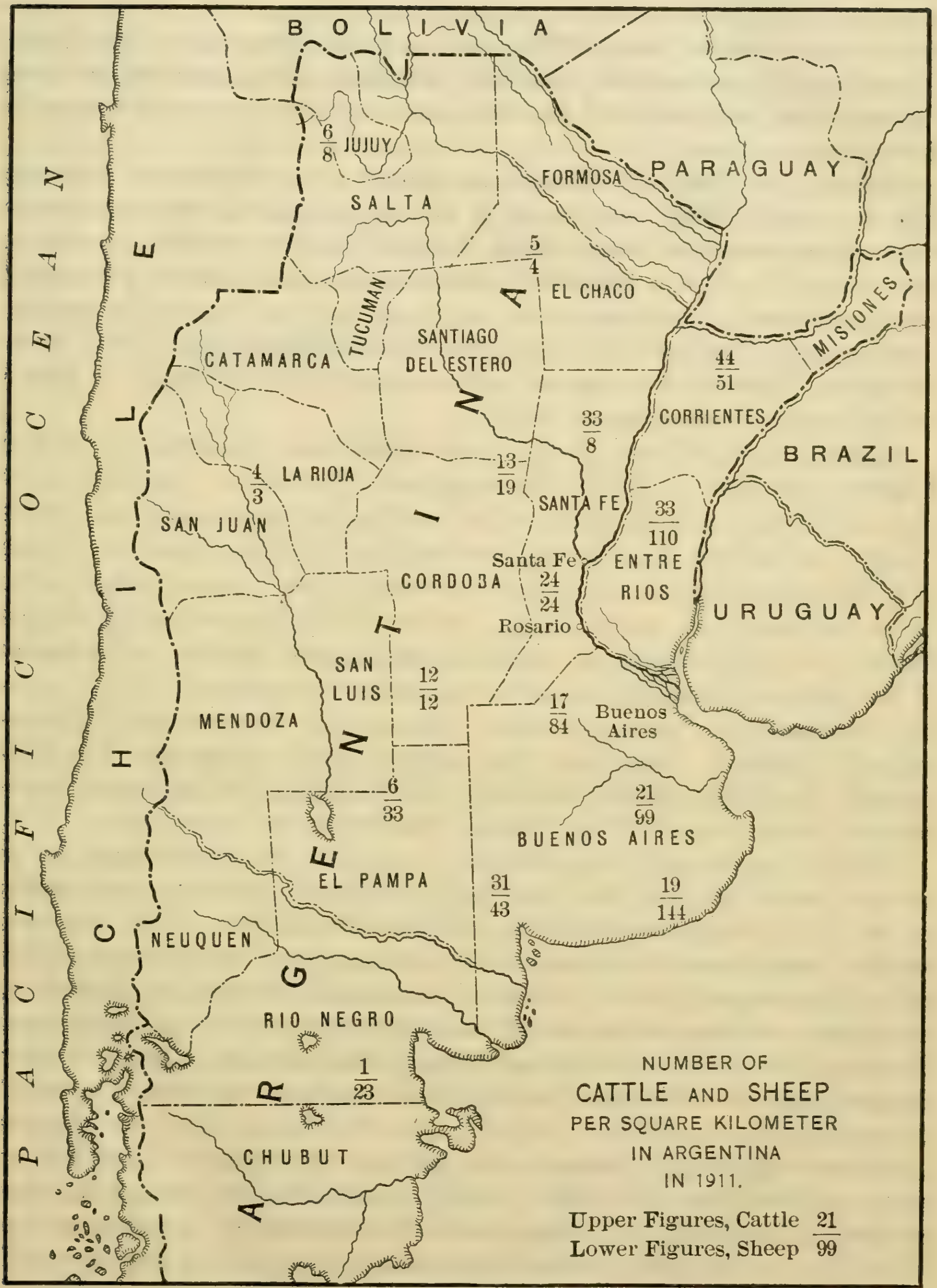
BOLIVIA
PARAGUAY
BRAZIL
URUGUAY
PACIFIC OCEAN
CHILE
ARGENTINA
JUJUY
SALTA
FORMOSA
EL CHACO
TUCUMAN
SANTIAGO DEL ESTERO
CATAMARCA
MISIONES
CORRIENTES
LA RIOJA
SAN JUAN
SANTA FE
ENTRE RIOS
CORDOBA
Santa Fe
Rosario
SAN LUIS
MENDOZA
Buenos Aires
BUENOS AIRES
EL PAMPA
NEUQUEN
RIO NEGRO
CHUBUT
6/8
5/4
44/51
33/8
4/3
13/19
33/110
24/24
12/12
17/84
6/33
21/99
31/43
19/144
1/23
NUMBER OF CATTLE AND SHEEP PER SQUARE KILOMETER IN ARGENTINA IN 1911.
Upper Figures, Cattle 21
Lower Figures, Sheep 99

FIG. 5.

The principal products of the three southerly States of Brazil, namely: Parana, Sta. Catharina, and Rio Grande do Sul, are timber, some agricultural products, and the yerba maté, of which latter thousands of tons are gathered annually and shipped to points all over the southern part of Brazil, Uruguay, and the Argentine. The development of this section will probably be largely along agricultural and pastoral lines, together with the exploitation of the forests of timber in the western sections.

Uruguay is likely to remain an almost entirely pastoral country, with some agriculture, and, as it is served by a fairly efficient system of standard-gauge railways which shows a healthy normal growth, it does not call for much detailed consideration in the present discussion, except perhaps to call attention again to the fact that this section, Uruguay, the Provinces of Entre Rios and Corrientes, and above them Paraguay, forming a connected area as shown by the map, is served exclusively by medium-gauge lines.

Chile, although one of the most important countries in South America, has little influence on the present discussion, inasmuch as it is cut off from the vast areas to the east by the high cordillera of the Andes. Its railway development was originally that of a series of lines from the mountains to the coast, and more or less perpendicular to the latter, although in recent years, under the influence partly of the scheme for a Pan-American railroad and partly for the development of its internal communications by a means independent of the coastwise steamers, it has put under construction the so-called "Longitudinal" railway, traversing the narrow strip along the foot-hills through the length of the country. The existing and proposed international connections of the Chilian Railroads are noted later.

Bolivia has been little developed, except for the exploitation of its mineral resources, and what development there is has been principally in the high mountain plateau to the south of La Paz. This has been reached from the south by the 2 ft. 6-in. line from Antofogasta, which, at its highest point (on a branch) reaches an elevation of 15 809 ft., the highest railway in the world, and from the north by the Southern Peruvian line (4 ft. 8½-in. gauge) from Mollendo and Cuzco. Within the past year the Arica-La Paz line (meter-gauge) has been completed, giving Bolivia an additional outlet to the Pacific, but all three of these lines have very steep gradients, and summit elevations of 15 000 ft. or more, so that they have only a limited capacity. It is expected that eventually there will be a considerable development of the eastern side of Bolivia, on the lower slopes of the mountains, and on the vast well-watered plains between them and the State of Matto Grosso in Brazil, and the outlet from this section of the country must inevitably be toward the Atlantic seaboard, either south through the Argentine, which if longer is, though much easier country, eastward through Para-

guay and Brazil to Rio de Janeiro and Santos, or northward through the Valley of the Amazon.

All this vast section seems to be entering on an era of development fostered by an ever-increasing tide of emigration from Europe, comparable only to that of the United States, and when it is considered—to take only one example—that the commerce of the City of Buenos Aires is growing at a rate at which it will probably be doubled in less than 10 years (the commerce of New York is doubled in about 20 years) it will be seen that the question of the policy of the future development of the railways is one of great importance. Figs. 6 and 7 show the increase in trade, exports, and imports, and the areas under cultivation for the three most important cereals.

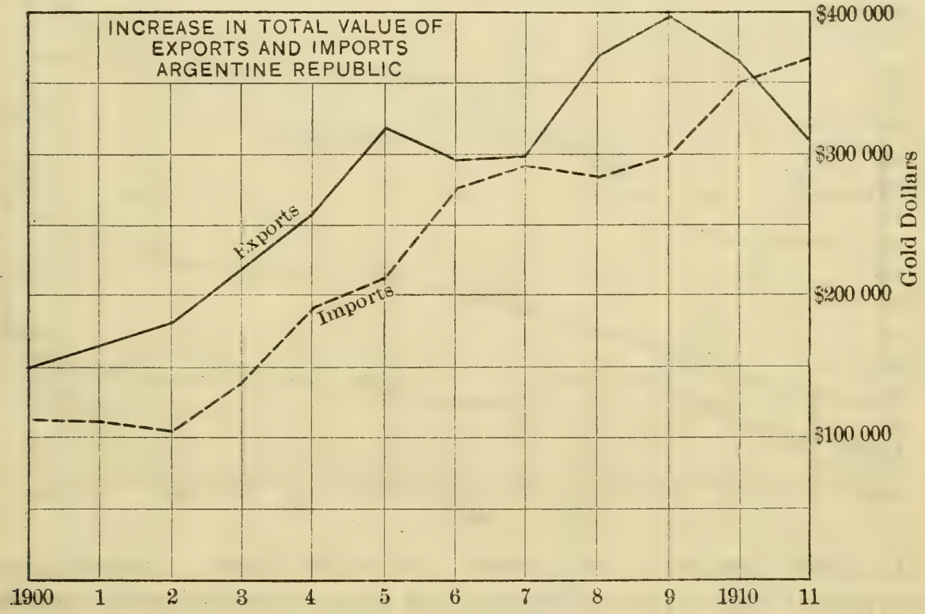

FIG. 6.

Immigration in the Argentine has increased from about 100 000 per annum in 1903 to 300 000 in 1910, by far the larger proportion being from Italy, Spain, and France, in the order named. The progress of railway construction is shown by Fig. 8. As will be noted, it has been rapid and steady, except for the setback following the crisis of 1890.

The latest official statistics of the Argentine Railways published are those for 1909, but Table 27, showing the length in operation, capitalization, earnings, etc., of the various lines for the year ending December 31st, 1912, is approximately correct.

The four broad-gauge lines are by far the most important. They have been built, and are controlled, by British interests. These lines radiate from the City of Buenos Aires, and have been most influential in the development of the country. They are so strongly intrenched

that they are a most important factor to be kept carefully in mind in any consideration of the question of gauge, as it seems entirely improbable that their owners and managers can be shown that there would be any benefit in changing.

The narrow-gauge lines of the Argentine have been principally developed in the section north of Rosario and Cordoba, and, until within comparatively recent times, partook more of the character of small local lines than of a connected system of trunk lines, none of them having reached the National Capital.

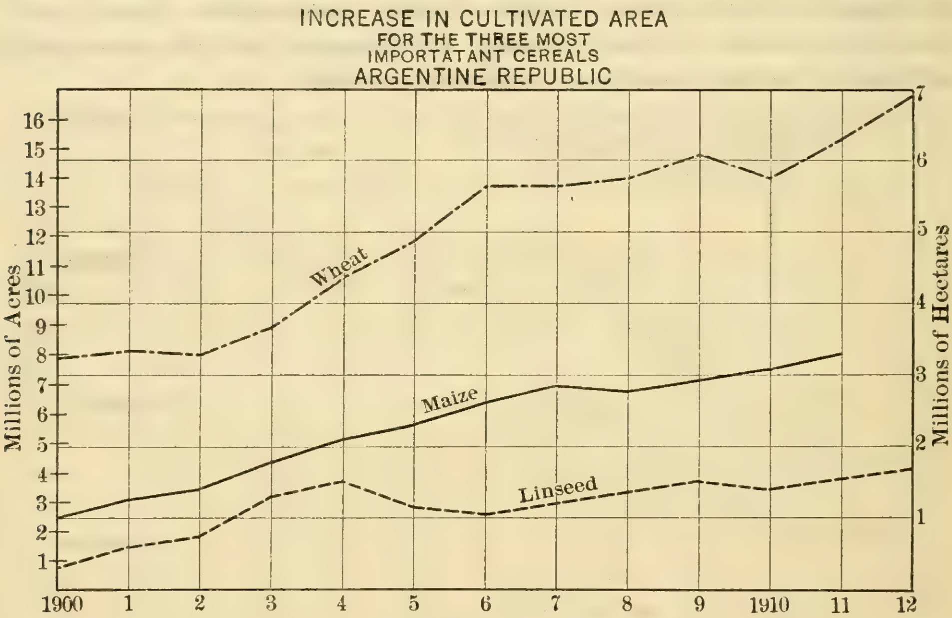

FIG. 7.

In 1908, however, some French capitalists built a narrow-gauge line from Buenos Aires to Rosario and there connected with the lines of the much older French company operating the Province of Santa Fé Railway reaching to Resistencia at the confluence of the Paraguay and Alto Parana, and connecting at Santa Fé with the Government lines through Tucuman to La Quiaca on the Bolivian border.

The Central Cordoba system, which had been formed by uniting some three or four small lines between Rosario, Cordoba, and Tucuman, with headquarters at Cordoba, decided about this same time to build an extension to Buenos Aires from Rosario, and thus connect its system with the National Capital, this line being opened to public service only about a year ago, and the headquarters of the system transferred to Buenos Aires. The Central Cordoba connects with the other part of the Government system, the Argentino del Norte, and also with the Central Norte at Tucuman.

The narrow-gauge lines are thus divided now into three groups: The French lines radiating in three directions from Buenos Aires and

extending north along the Parana up to Resistencia; the Central Cordoba starting from Buenos Aires and passing through Rosario and Cordoba to Tucuman, and the Government lines reaching all the capitals of the northern and Andean provinces and scattered along the foot-hills of the mountains between San Juan and Bolivia, connecting at Cordoba and Santa Fé with the other lines and thus having through communication with Buenos Aires.

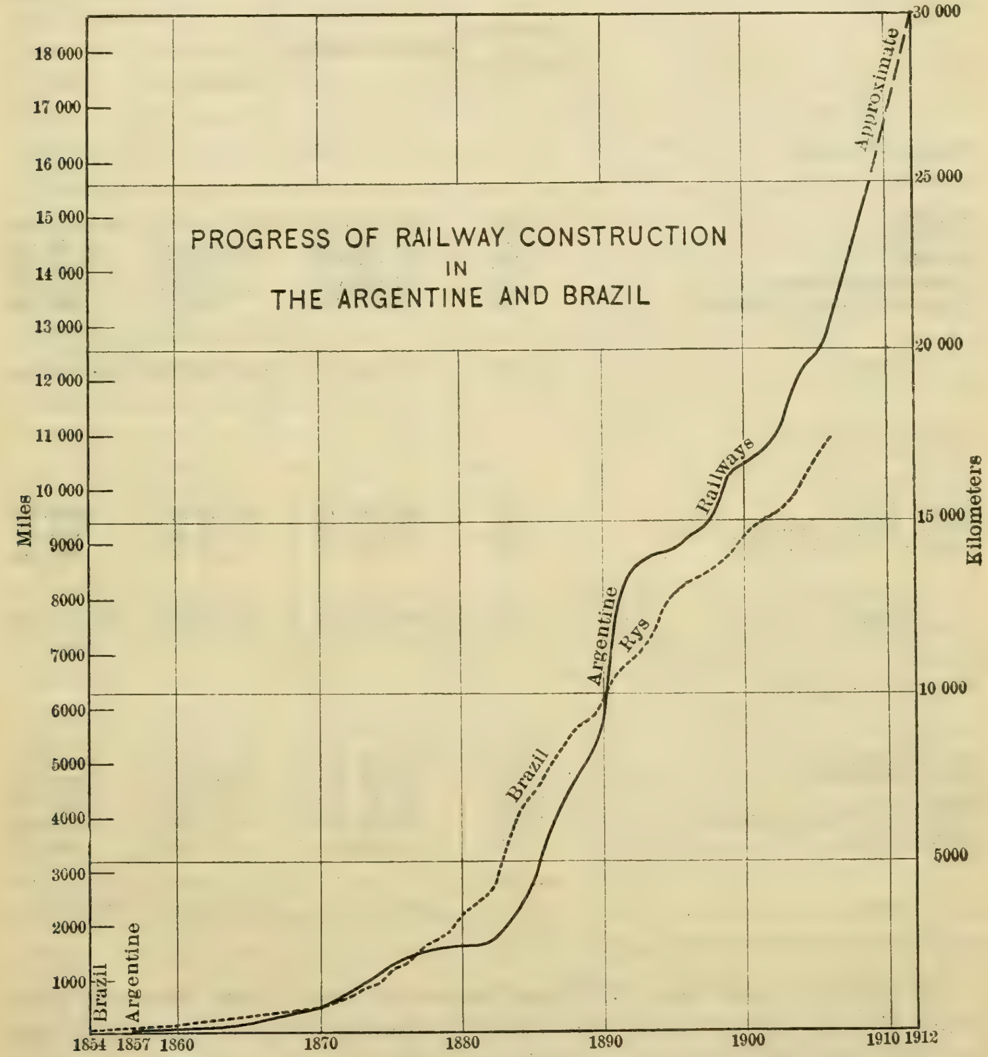

FIG. 8.

These conditions on the narrow-gauge lines must be kept well in mind, as they are just now emerging from a status as more or less isolated lines, doing a small local business, to that of a more or less coherent group of systems of trunk lines connecting the National Capital with the northern section of the country, and reaching nearly all the provincial capitals. As local lines doing a small business, the gauge in itself mattered little, but now, with the greatly increased

business that is coming to them, and the growing necessity of running through express trains at fairly good speeds, they find themselves handicapped by reason of this very increase in business, because they have not the facilities for doing it properly.

TABLE 27.

	Length, in kilometers.	Capitalization.	Earnings.	
			Gross.	Net.
Broad-Gauge.				
Southern	5 608	$220 503 600	$27 127 561	$11 311 439
Western	2 669	101 959 200	12 225 441	5 474 275
Pacific	5 342	216 086 900	24 405 713	9 185 113
Central Argentine	4 751	197 276 600	26 360 005	11 481 579
Rosario á Puerto Belgrano	794	30 957 100	553 575	903 958
Totals	19 164	$766 783 400	$90 672 295	$38 356 364
Medium-Gauge.				
Entre Rios	1 175	$30 391 700	$2 379 390	$908 096
North East Argentine	1 074	29 538 100	1 609 335	622 090
Central of Buenos Aires	269	8 942 800	977 091	305 389
Totals	2 518	$68 872 600	$4 965 816	$1 835 575
Narrow-Gauge.				
Government lines	4 018	$121 872 900	$6 292 069	$359 428
Central Cordoba	1 935	70 525 000	8 220 351	2 253 352
Santa Fé	1 709	42 131 700	5 787 433	2 085 278
Province of Buenos Aires	1 267	39 399 300	2 497 010	527 847
Transandine	185	8 902 800	676 605	125 299
Totals	9 114	$282 831 700	$23 473 468	$5 383 204

It will be noted that the broad-gauge railways have through lines from Buenos Aires to Rosario, Cordoba, Santa Fé, and Tucuman, and, by reason of their long establishment, better track, and more comfortable, because easier-riding, rolling stock, have up to the present monopolized the through passenger business to and from these points. In order, therefore, to get their share of this business, the narrow-gauge lines will be forced to make considerable improvements in their service, starting with the physical condition of their roadbed and track, and working up and even then, of course, can never compete on even terms.

It is often pointed out, in discussing the gauge, that these narrow-gauge lines have done very well, that their capitalization is small and, therefore, there is no reason to change, but, as will be shown later, it seems to the writer that they must look forward to the growth of the country and be prepared to meet it by modern economic means of transport or be crowded out. By the time they have provided adequate terminal facilities in the large cities, their capital expenditures will not be so much less than that of the broad-gauge lines, and they can never, of course, handle the same amount of business.

To convey some idea of the importance of the broad-gauge lines, Figs. 9, 10, and 11 show parts of the lines of the Southern, Pacific, and Central Argentine Railways, where they enter the City of Buenos Aires over expensive steel and masonry viaducts. They have extensive and expensive terminals, both passenger and freight; the Southern has four tracks for some distance out of the city. Fig. 12 shows the Southern Railway Passenger Station. The Central Argentine has nearly completed its second track to Rosario; the Central Argentine and Western have plans under way for the electrification of their suburban zones; the Central Argentine is building an extensive new terminal and station; the Western is building a tunnel connection to the Port Railway through the heart of the City of Buenos Aires, and nearly all have a considerable portion of their main lines stone-ballasted with stone at $2 to $3 (gold) per cu. m. f. o. b. cars ($1.50 to $2.50 per cu. yd.).

The Provinces of Entre Rios and Corrientes, within the confines of which are located the two principal medium-gauge railways, are almost entirely surrounded by the Rivers Parana and Uruguay. They are thus practically cut off entirely from communication with the rest of the Argentine, or indeed with any other part of the world, except by water transportation. Their isolated position, therefore, has rather retarded their development, in spite of the richness of the lands in many parts, and the railways have been merely local lines from the interior to the small ports along the rivers.

About 1906, however, it was decided to effect a connection with the City of Buenos Aires by an extension of the line to Ibicuy and a car ferry across the delta of the Parana River to Zarate, a small town in the Province of Buenos Aires. Here connection was made with the Central of Buenos Aires Railway, then a rural steam tramway of medium-gauge, over which running rights were obtained into the City of Buenos Aires, though at a point a fairly long distance out from the center. This through line was opened to public service only about 4 years ago.

At about the same time, also, the Central Railroad of Paraguay, which at that time was a broad-gauge line running from Asuncion, the capital of that country, to Villa Rica in the interior, was changed

to medium-gauge and extended to Encarnacion, opposite Posadas in Corrientes, and another car ferry established there, so that by these two car ferries, through train service between the City of Buenos Aires and Asuncion, through the Provinces of Entre Rios and Corrientes is now possible, thus putting all this section in close touch with Buenos Aires and the outside world.

It will be seen, therefore, that the whole railroad situation in the Argentine has undergone a radical change during the past 5 years or so. Previous to that time, the four large broad-gauge systems controlled by British interests, which in many respects were identical, dominated almost exclusively the transportation of the country, and the lines of other gauges were of little importance. Now the medium-gauge lines are not only linked together, but are all (except the Central of Buenos Aires) controlled by mutually friendly interests with a through route some 900 miles long from Buenos Aires to Asuncion. The narrow-gauge lines are linked up into not more than three systems, all of which are working together in harmony and may come under a single control. They reach the northern section of the country and the one in which the great development of the next 20 years is to take place. The question is, shall this development be by means of a system of narrow-gauge lines, can such a system offer adequate transportation facilities, and can it offer competition to the broad-gauge lines, should they decide to extend their lines into this section, and with river transportation on the Parana?

There is also to be considered the question of international connections, and the present and rapidly increasing interest in South America, both by Europe and North America, makes this important. To the writer the idea of a Pan-American Railroad as a continuous trunk line for the transportation of through freight and passengers from the United States and Canada to the extreme south, has always seemed almost chimerical. The variation in gauges, as will be seen, is an important obstacle, and such a line can never hope to compete for long-distance transportation through a tropical climate and across the vast and lengthy stretches of the most rugged mountains in the world, with the facilities offered by ocean steamers.

It is easy, of course, to foresee that at some time in the future, which seems now not a little distant, the various lines of railway will be linked together, and that there will be continuous connections. This, however, will not be, at least in the writer's estimation, until there is a local demand and local business sufficient to warrant the construction of the various constituent parts. It is also entirely probable that such a line will be useful and will be used for international communications, between any two or more countries within reasonable limits, and for this reason uniformity of gauge should be advocated throughout the continent and through the Central American zone,

FIG. 9.—OLD STEEL VIADUCT, CENTRAL ARGENTINE RAILWAY, ENTERING BUENOS AIRES.

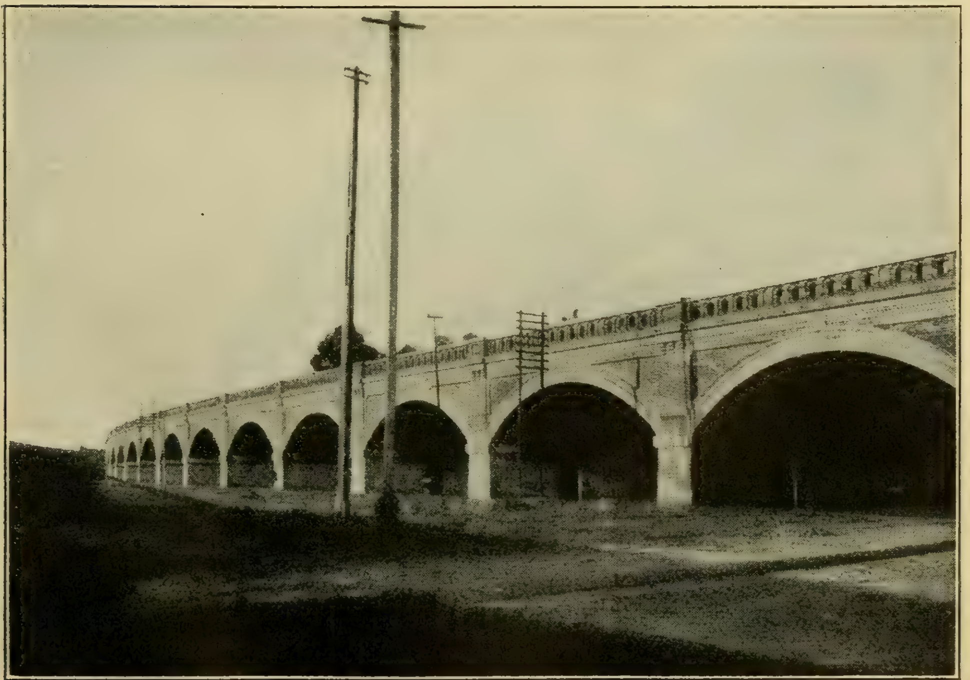

FIG. 10.—MASONRY VIADUCT, PACIFIC RAILWAY, ENTERING BUENOS AIRES.

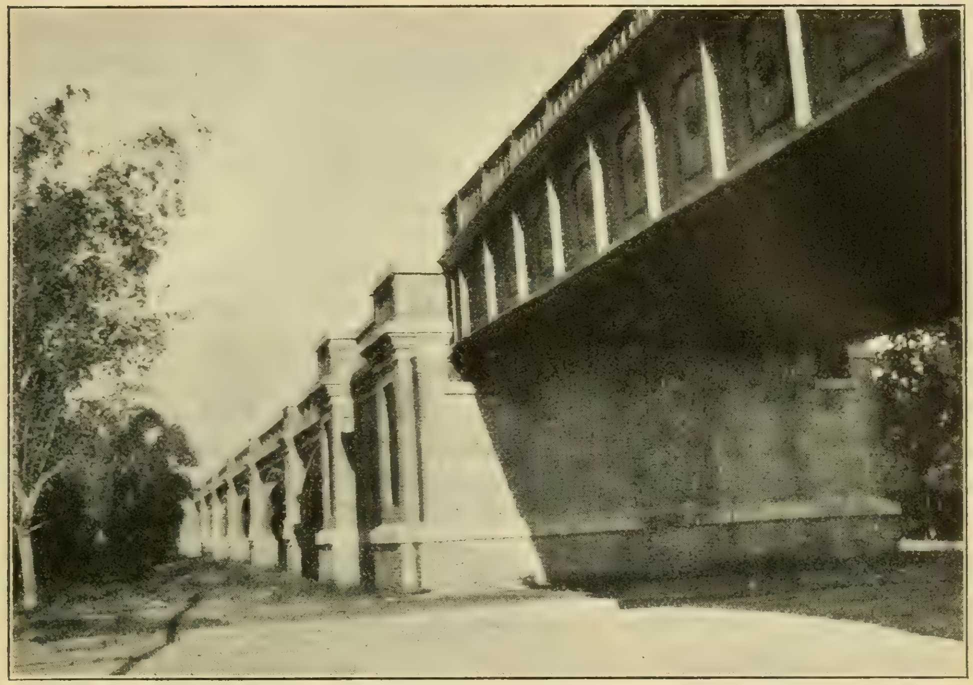

FIG. 11.—STEEL BRIDGE AND MASONRY VIADUCT, CENTRAL ARGENTINE RAILWAY, ENTERING BUENOS AIRES.

FIG. 12.—SOUTHERN RAILWAY PASSENGER TERMINAL STATION, BUENOS AIRES.

this question being one which should be most energetically pressed at any further Pan-American Congresses and by the representatives of the Pan-American Union.

It has been previously pointed out how, in the Argentine, the four big broad-gauge systems radiating from Buenos Aires dominated the situation up to within 5 or 6 years ago, and how, since then, the other systems have formed coherent groups with connections to the National Capital. During the same period, also, an international syndicate of bankers has acquired control of the railway lines in the three States of Southern Brazil, and a substantial interest in those in the southerly half of the State of São Paulo. This same syndicate also has operating control over three lines in the Argentine, the Rosario á Puerto Belgrano, 5 ft. 6-in. gauge, the Central Cordoba, meter-gauge, and the Entre Rios, medium-gauge, it has established a community of interest with the North East Argentine Railway, which connects the Entre Rios Railway with the Paraguay Central Railway, which latter it also controls. It has also a substantial interest in the Antofogasta Railway, in Chile and Bolivia. In addition to its railways, this syndicate is taking great interest in the exploration, exploitation, and development of the countries served by its lines and those beyond into which they may be extended.

One of the lines controlled by these interests has already been pushed across the State of São Paulo, crossing the Alto Parana nearly 1 500 miles above its mouth, and, eventually, without doubt, will be pushed ahead through the State of Matto Grosso into Eastern Bolivia and to Sucre on the headwaters of the Amazon. Another line is projected along the boundary line between the States of Parana and Sta. Catharina, following the Valley of the Iguazu to its confluence with the Alto Parana, crossing the latter at a point some 500 miles below that of the line into Matto Grosso, just referred to, and connecting with the line now under construction eastward from Villa Rica on the Paraguay Central. This will give a through route from Rio, São Paulo, and all points in Southern Brazil to Asuncion, the capital of Paraguay, though there will be a break of gauge at the border. From Asuncion this line will probably be extended northwestward between the Pilcomayo and Paraguay Rivers toward Sucre.

There is another east and west line still farther south in Brazil from Porto Alegre to Uruguayana, a point on the River Uruguay opposite Paso de los Libres on the North East Argentine Railway, where connection can be made either to Santa Fé or Resistencia on the west side of the Parana, and from these latter places to all points in the northern section of the Argentine and to Bolivia. This latter route at present involves more or less delay, steamer transportation across the two rivers, and two breaks of gauge, but eventually this will be developed into a through route.

There is already through rail connection between Rio and Montevideo, with breaks of gauge, however, at São Paulo and at the border of Uruguay and Brazil. At least two schemes have been proposed for a short through line from Colonia, a point in Uruguay opposite Buenos Aires, to Rio, over which passengers can be carried in express trains at reasonably fast speeds, and thus shorten the length of the sea trip between the Argentine and Europe, and the time between Buenos Aires and Rio by nearly half. This, if it is ever realized, means a standard-gauge line through Southern Brazil, as the Uruguayan gauge is fixed and it would be out of the question to get the necessary degree of speed and comfort on any lesser gauge to compete with the fast, luxurious ocean steamers in which the trip can now be made and which are being improved every day. Perhaps present conditions hardly warrant the construction of this line as yet, but that it is a development of the not far distant future is hardly open to question.

There is, as is perhaps well known, one line which now crosses the Andes, giving direct rail connection between Buenos Aires and Valparaiso. This, however, involves a change at the break of gauge at Mendoza in the Argentine and at Los Andes in Chile, the Buenos Aires and Pacific is 5 ft. 6-in. gauge, the Transandine in the Argentine and Chile is meter-gauge, and the Chilean lines with which it connects are 5 ft. 6 in. Another connection is proposed in the south by the extension of the Southern Railway's (5 ft. 6 in.) line from Neuquen, and there is a proposal (which, however, it does not seem will be carried through in the immediate future) to build a line from Salta in the Argentine to Mejillones in Chile. Farthern north, the Antofogasta Railway has been building an extension southeastward from Uyuni toward Tupiza, and probably before long it will be connected with the lines of the Argentine Government (meter-gauge) at La Quiaca.

Turning now to the Argentine: The area west and south of Bahia Blanca, it seems, must inevitably be developed by the Southern and Western Railways, that is, by the 5 ft. 6-in. gauge, and the country north of Santa Fé and Cordoba is the part in which may be expected the most important developments of the future which will be of interest to this discussion.

The line of the Central Cordoba Railway from Cordoba to Tucuman, and the Government line from Tucuman by Salta to Embarcacion may be taken as marking the dividing line between the mountains and the vast Argentine plain. Between Santa Fé and Tucuman there are two lines, the more southerly being the Central Argentine broad-gauge, and the more northerly the Central Norte, meter-gauge, operated by the Government. It is the area north of this latter line and east of the line from Tucuman to Embarcacion, and known generally as the Gran Chaco, in which the development of new territory by new

lines will probably be most active in the Argentine. The topography of this region is almost as blank as the space shown on the map, and its character does not change for at least some distance north of the Pilcomayo in Paraguay and Bolivia. The distance from Santa Fé to the northerly border of this region is about 750 miles, and from Asuncion to Tucuman about 550 miles, and it is probably safe to assume that railways may be built at a minimum cost and in practically straight lines over almost any portion of it.

Santa Fé is a port at present available at lowest river stages for ocean-going steamers drawing 18 ft. of water, and this depth will probably be increased in the near future. It seems to the writer improbable that there will be a port for ocean-going steamers established north of this point, so that the commerce of this vast region must come to Santa Fé. It is true that there may be some competition by way of the rivers, but experience elsewhere has shown that this will not be a serious matter for properly equipped railroads, though it might be for lines any less efficient than those designed in the light of the best present knowledge of the art. This area must be served by railroads of some sort, they must be efficient if they are to pay and be able to compete with water transport.

The Argentine Government has commenced the development of this region by lines running more or less westward from points on the Parana River at Resistencia and Formosa, apparently with the idea of bringing the products from the interior to the river and thence by fluvial transportation to the coast, the merchandise and supplies returning the same way. This, the writer believes to be fundamentally wrong, and that this area might better be developed by a series of north and south lines from Santa Fé and points on the existing lines between Santa Fé and Tucuman, which will reach up into the north beyond the Pilcomayo and bring all this vast region in touch with Buenos Aires. Here it may perhaps be well to refer again to the importance of the City of Buenos Aires as a distributing point for all merchandise, its dominating influence over the whole country as the seat of the national government, and the absolute importance of proper and adequate means of transportation to reach it from all parts of the country.

A certain number of east and west lines between Brazil and the north, along the lines already being developed, and as already indicated, will be necessary of course for the interchange of products and for communication between the large populations which will, in all probability, occupy this area before another quarter of a century has passed, but the line of least resistance to and from the southern part of eastern Bolivia, western Paraguay, and the Chaco is by the most direct north and south routes between this section and Buenos Aires.

The exact details of the lines along which the future development of this area may proceed, however, are not of particular importance in this discussion. The outline of the general scheme which has been presented will at least tend to show its close resemblance to that of the United States during the time our great West was being opened up to agricultural developments. Knowing as we do the important role played by the railroads in this era, it seems obvious that a system of transporation less efficient than our own in moving bulk freight long distances at low cost would handicap the development of the great area in the Southern Hemisphere.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

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SUMMARY.

As bridges of exceedingly long span are needed these days in a number of places, it has become necessary to use alloys of high elastic limit and great ultimate strength, in order to lessen materially the dead load to be carried. Up to the present time, nothing more satisfactory or effective than nickel steel has been found; and even that alloy has not been developed to the limit of its practical possibilities. This is due solely to a reluctance on the part of manufacturers of bridge metal to take any special pains in its production.

The time has come to discover some alloy of steel (or some other metal) of great strength, which, in every detail, will be suitable for undergoing the manipulations required of bridge material during the various processes of fabrication.

The objects of this paper are:

First.—To show, for the different span lengths, what the weights of metal per linear foot of bridge would be, in both simple-truss and cantilever structures, when using metals of various elastic limits, ranging from that of ordinary carbon steel up to 100 000 lb. per sq. in.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Second.—To indicate the extreme practicable limits of span length for cantilever bridges constructed for the greater part of such materials.

Third.—To demonstrate the comparative economics of finished bridges involved in using metals of the several assumed elastic limits, for all reasonable excess costs of metal per pound, delivered at site, as compared with carbon steel structures.

In the diagrams which form a part of the paper, Figs. 5 and 6 show the weights of metal per linear foot of structure for double-track railway bridges of simple-span and cantilever construction, beginning with short spans and extending to the limits of utility. Directions are given in the text for finding the weights of metal per foot for bridges having any number of tracks and for those designed with other live loads than those adopted in the preparation of the paper. From these diagrams and from the text it will be seen that the possible limits of main span length for cantilever bridges vary from some 2 000 ft. for carbon-steel structures to about 3 500 ft. for those built of metal having an elastic limit of 100 000 lb. per sq. in.

Figs. 10 to 21, inclusive, are diagrams showing the costs of metal erected per linear foot of bridge, in both simple spans and cantilevers, for the different elastic limits assumed. There is a curve for each excess pound price of metal work, delivered at site, above that of carbon steel; and these excesses vary by ½ cent. per lb., and extend always slightly beyond the greatest increase that need be considered. On each diagram there is shown, by a line heavier than the others, the cost of erected metal per foot of structure for carbon-steel bridges under the conditions assumed, so as to afford a means of instantaneous comparison of costs between any all-carbon-steel bridge and the corresponding structure built chiefly of the metal to which the diagram pertains.

Figs. 22 and 23 show the comparative economics of all the alloy steels (or other metals) considered, under the assumption that their pound prices for material delivered at the bridge site are the same.

In his paper on "Nickel Steel for Bridges",* the writer showed the results in bridge construction obtainable by the use of nickel steel, and although the limits of attainment indicated therein have not yet

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 101.

been reached by American bridge builders, because the metal manufacturers are not willing to guarantee an elastic limit of 60 000 lb. per sq. in. for that alloy, nevertheless, since its publication, a number of important structures have been built of nickel steel, and the manufacturers have indicated to the engineers a willingness to increase their set elastic limit of 50 000 lb., provided they be satisfactorily compensated. The obstacle in the way of adopting a 60 000-lb. elastic limit is the fact that there is no market for the rejected material. The writer is of the opinion, however, that, after a little experience, the manufacturers would have no more trouble in furnishing the alloy with an elastic limit of 60 000 lb. than in furnishing ordinary carbon steel with an elastic limit of 35 000 lb. The best practicable solution of the difficulty is an agreement by the purchaser to pay the manufacturer a fair compensation for his loss by rejections. The writer's reason for thinking that no great trouble will be encountered in manufacturing nickel steel with a minimum elastic limit of 60 000 lb. is that the inspection sheets for the Free Bridge over the Mississippi River at St. Louis, engineered by the late Alfred P. Boller and Henry W. Hodge, Members, Am. Soc. C. E., seldom show an elastic limit materially less than that, although the requirement fixed by arrangement with the manufacturers was only 50 000 lb.

It is the writer's intention, on the first opportunity that presents itself to him for building a bridge of very long span, to construct it of mixed nickel and carbon steels, placing the alloy wherever it can be used to advantage, and arranging, if possible, with the manufacturers for a minimum elastic limit of 60 000 lb., so as to show actually what can be accomplished by adopting nickel steel of that strength, and complying with the specifications given in the before-mentioned paper on "Nickel Steel for Bridges".

A condition which at present militates seriously against the use of nickel steel in bridge building is that the manufacturers ask for it an additional price of some 2 cents per lb., as compared with ordinary carbon steel, although but little more than one-half of that would be a sufficient excess price. This, undoubtedly, is because structural steel manufacturers naturally object to fundamental innovations, preferring to follow lines of least resistance; but, as a number of very long steel spans are contemplated for the near future, they will certainly be forced sooner or later into using high-alloy steels.

Again, it has been stated authoritatively to the writer by Mr. T. L. Willson, the eminent Canadian authority on thermo-chemistry, that ferro-nickel, containing 10% of nickel and nearly 90% of iron, can be furnished with a good profit at 2 cents per lb. for adding to the molten carbon steel in the manufacture of nickel steel, and that no trouble would be involved in burning out all the impurities contained in the said ferro-nickel. This would make the nickel content in the alloy cost about 10 cents per lb. instead of 30 cents, as assumed in the before-mentioned paper. The economics involved by such use of ferro-nickel will be treated hereinafter.

During a recent conversation with two of the largest European producers of nickel, the writer was told that it would be practicable for them to furnish pure nickel in large quantities for the purpose of manufacturing bridge material and to sell this pure nickel at about 20 cents per lb.; but they dropped the hint that they were in no hurry to do so, as the demand for that metal to-day is in excess of the supply, and the main obstacle in the way of large production is likely to be a scarcity of satisfactory nickel ores.

When in France in 1909, the writer's attention was called to the fact that certain metal manufacturers in that country were making, in melts of 5 tons and less, a purified carbon steel, for which rather astonishing claims, in regard to great ultimate strength, high elastic limit, and general suitability for the manufacture of bridges, were made; and on his investigating the matter by both interview and correspondence, he was convinced that these claims might, at least partly, be justified by performance. Thereupon, having some spare time, he prepared an economic study of the possibilities for utilizing such purified steel in bridges, using in his calculations French units, prices, and other conditions, and published the results in French,* under the title, *"Étude Économique de l'Emploi de l'Acier au Carbone à Grande Résistance, pour la Construction des Ponts."*

The excess cost of this purified steel, as compared with ordinary carbon bridge steel, was claimed to be a little less than 1 cent per lb. for the manufactured bridges; and the investigation showed the economics of its use in bridge building for the mean and the extreme conditions of the French metal market, and for a number of elastic limits varying

* *Le Génie Civil*, August 7th, 1909.

from 30 to 45 kg. per sq. mm., the value for French carbon bridge steel being 24 and that for the writer's specified nickel steel 42.5 kg. per sq. mm. The result of the study indicated that there was no advantage for the 30-kg. elastic limit; none for short spans but a small one for long spans with a 35-kg. elastic limit; a decided saving for all cases with a 40-kg. limit; and a wonderful economy for the 45-kg. limit, the highest elastic limit claimed by any of the French manufacturers.

The writer had hoped that that paper would give an impetus in France (and perhaps elsewhere also) to the manufacture of bridges of purified steel; but, up to the present time, he has not heard of any such development. It is either that the claims of the manufacturers have not been justified by performance, or that the same conditions of inertia and *laisser aller* exist in Europe, in respect to innovations in the manufacture of new bridge metal, as govern in America in relation to the adoption of nickel steel for bridges.

As the future development of America will necessitate the building of many very long-span bridges, it is almost a necessity that there be found an alloy of steel of great strength, high elastic limit, workable under all necessary manipulations in the shops, and of moderate cost. Such an alloy is not going to be discovered by accident, but only by a lengthy and exhaustive series of experiments, laid out systematically in advance, and modified from time to time as knowledge of the subject is accumulated. These experiments should be performed where small melts of steel are readily procurable, and where the experimenter will not be unduly delayed by the metal manufacturers. These conditions are found in France—for there one can procure single ton melts, and possibly smaller ones, if necessary. Arrangements could be made with the mills for rolling into plates and shapes the metal from the various melts without delay; and the experimenter could have at his disposal, not only a first-class testing machine, but also a bridge shop, where all the usual manipulations of the metal might be made. By experimenting at first on different kinds of alloy materials and later on mixtures thereof, one eventually should discover some satisfactory combination that would afford a suitable material of great strength at comparatively moderate cost. The more elaborate and thorough the experiments, the greater the probability of discovering a truly suitable steel of exceedingly high resistance at a reasonable expenditure for production and manufacture.

Such a series of experiments would be expensive, costing fully $100 000, and possibly twice that amount; but the saving in cost on one big bridge alone might easily exceed the entire expenditure. The money required for such experiments could not be obtained from bridge manufacturers, because it is not to their interest to inaugurate changes; but it might be secured from some millionaire philanthropist or from a group of capitalists who contemplate the construction of some great steel bridge.

To show the possibilities for long-span bridge construction by the use of such superior alloys of steel, and the economics thereof, is the object of this paper.

The basis of the following investigation is a mass of diagrammed and tabulated data concerning weights of metal in simple-span and cantilever bridges of carbon steel, up to a limit of 600-ft. spans for the former and 1 800-ft. main openings for the latter, accumulated by the writer and his firm during the last quarter of a century, and the weights of nickel-steel bridges and of mixed nickel-steel and carbon-steel bridges computed by the writer in the preparation of his before-mentioned paper on "Nickel Steel for Bridges". Because the said diagrammed weights of metal per linear foot of span in simple-truss bridges are limited to lengths of 600 ft. it has been found necessary, in preparing this paper, to extend them to 1 000 ft. by making actual calculations of stresses, sections, and weights of metal for several long spans, using the various kinds of steel assumed. From these recorded and specially computed weights, and by the succeeding formulas of reduction and extension, have been prepared the various diagrams of weights of metal per linear foot of span contained in this paper.

Many preliminary and tentative diagrams of curves were constructed, but, for considerations of space and cost, they are not included herein. These individual diagrams, prepared for each class of steel and each type of bridge, divided the weights into those for "Floor System"; "Lateral System"; "Trusses"; and "On Piers"; but in the selected diagrams only the total weights of metal per linear foot of span for a combination of all the parts of the structures are recorded, as these quantities constitute the results sought, and to be used in the succeeding economic investigation. Near the end of this paper, however, are given some tables from which can be found the approximate values of weights of metal per linear foot of span for "Floor

Systems"; "Lateral Systems"; and "On Piers"; and from these and the diagrams, Figs. 5 and 6, can be computed the weights for "Trusses".

The weights given for bridges of carbon steel are based on the standard specifications for designing contained in the writer's "De Pontibus". They are quite accurate up to the before-mentioned limits of 1 000 ft. for simple spans and 1 800 ft. for the main openings of cantilever bridges. Beyond these limits (shown on the diagrams by dotted lines) they are to a certain extent conjectural, although, in all probability, not far from correct, because of the thorough consideration which has been given to all factors influencing their values.

The weights for steel having an elastic limit of 60 000 lb. per sq. in. are those obtained for the diagrams of the paper on "Nickel Steel for Bridges", and are based on the specifications contained in that paper. They are quite accurate up to the before-mentioned limits of 1 000 ft. for simple-truss spans and 1 800 ft. for the main openings of cantilever bridges; and a comparison of the curves for cantilevers of openings from 1 200 to 2 600 ft., shown in Fig. 71 of the writer's Nickel Steel paper with those of Fig. 6 of this paper indicates, in general, an excellent agreement. In the former the curves beyond 1 800-ft. openings were determined by an assumed regularity of continuity, as explained in that paper, and in the latter they were computed by the use of Equation 19 of this paper.

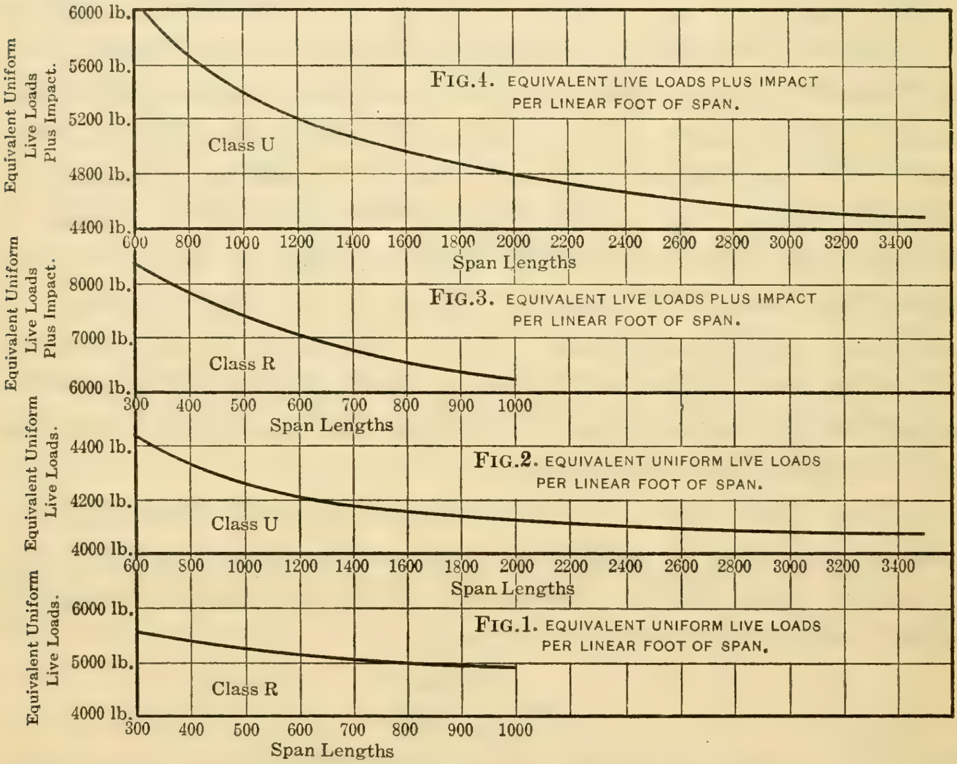
In Figs. 1 and 2 are shown the equivalent uniformly distributed live loads per linear foot of single track assumed in computing the weight of trusses. The impact percentages used were obtained from the writer's formula in "De Pontibus", *viz.:*

$$I = \frac{40\,000}{L + 500},$$

where I is the percentage of impact and L is the length, in feet, of the portion of the bridge which must be covered by the moving load in order to obtain the maximum stress on the member under consideration. Although it is true that the writer has evolved a more accurate formula than this, based on some lately made experiments on impact in railway bridges from moving loads, he has decided to adhere to the old formula so as not to necessitate the changing of the weight curves on which this investigation is primarily founded. Figs. 3 and 4 give a combination of the equivalent uniform live loads and the impacts therefrom. The loads obtained from these curves added to the dead loads

make the total loads per linear foot on the bridges, from which total loads the load ratios of reduction are found.

The following are the formulas of reduction used in passing from known weights of metal per linear foot of span in carbon-steel bridges to the corresponding weights in alloy-steel bridges. An observation of the nomenclature will show that the unaccented capital letters severally represent weights of metal per linear foot of span in carbon-steel bridges (or otherwise known weights of bridges of any kind of steel), and the accented capital letters, the corresponding weights for alloy-steel bridges (or otherwise the corresponding unknown weights of



bridges of some other kind of steel), also that the small letters severally represent linear dimensions of structures, the main exceptions being that capital R is used for reactions and small r for ratios.

Floor System.—

Let F = weight of metal per linear foot of span in the "Floor System" of carbon-steel bridges.

L = ditto for "Lateral System".

T = ditto for "Trusses".

P = ditto for "On Piers", including anchorage material in the case of cantilever bridges.

A certain portion of the weight of the floor system will vary inversely with the elastic limit of the steel, and the remainder will be invariable.

Let V = the variable portion,
and I = the invariable portion.

Then $F = V + I$. . . . . (1)

Let F' = the weight of metal per linear foot of span in the floor system of alloy-steel bridges,
and r (greater than unity) = the ratio of elastic limits of alloy steel and carbon steel,

Then $F' = I + \frac{V}{r}$ . . . . (2)

In heavy, double-track bridges, and especially those of long span, I will be approximately 0.35 F, and V approximately 0.65 F, hence

$$F' = 0.35\,F + \frac{0.65\,F}{r} = F\left(0.35 + \frac{0.65}{r}\right) \qquad (3)$$

In dealing with spans of greater length than any of those yet actually computed, it must not be forgotten that the increasing width of structure will augment the weight of the floor-beams and, consequently, the weight of metal per linear foot of span for the floor system. In case of double-track bridges, an economy can be effected by widening the cantilever arms and the anchor arms uniformly from ends to supporting pier; but it is probable that motives of policy would lead the projectors to construct exceedingly long spans so as to carry more than two tracks.

Lateral System.—

Let l_1 = length of span at which it pays to begin to use high steel for the laterals beyond the ends of l_1, it being assumed that the weight of laterals is uniform over the entire length, l_1, or, in other words, that minimum sections are used therein throughout;

R_1 = wind reaction at end of l_1;

R = wind reaction at end of span l;

r_w = ratio (greater than unity) of R and R_1;

L_1 = weight of carbon steel per linear foot for lateral system over the length, l_1;

L'_2 = weight of mixed carbon and alloy steels per linear foot of span at end of span, l:

$$\text{Then } L'_2 = L_1 \left(0.3 + 0.7 \frac{r_w}{r}\right).$$

Let L'_a = average weight of metal per linear foot for entire span, l,

$$\text{Then } L'_a = \frac{1}{l} \left\{ L_1 \ l_1 + \frac{L'_2 + L_1}{2} (l - l_1) \right\} \ldots\ldots\ldots (4)$$

Should L'_2 figure less than L_1, it shows that near the ends of the span minimum sections of the high steel must be used and that L'_a will equal L_1.

In passing beyond the limits of actually figured spans, when computing the weights of metal in lateral systems, it must be remembered that, as just explained for the floor system, the weight per foot is increased, not only because of the greater span length but also because of the greater span width. As a rule, it may be stated that, for any very long span (the length thereof remaining constant), the effect of increasing the width between central planes of trusses $n\%$ is to increase the weight of metal in the lateral system about $\frac{n}{2}$ per cent.

Trusses.—In respect to the weight, T, of metal per linear foot of span for trusses of carbon steel, the following equation may be used:

$$T = K + T_1 + C_c + C_w \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where K is the portion of the total truss weight per linear foot which is independent of the quality of the metal and of the stresses; T_1 is that of the main portions of the tension members and of their details that are directly affected by the stresses; C_c is that of the main portions of the compression chords and inclined end posts and their details that are directly affected by the stresses; and C_w is that of the main portions of the compression web members.

From experience in designing large bridges it may be stated that, as an average,

$$K = 0.2T$$
$$T_1 = 0.3T$$
$$C_c = 0.3T$$
$$C_w = 0.2T$$

Both T_1 and C_c (and consequently their sum) will vary inversely with the elastic limit of the metal; but C_w, on account of the influence of the ratio of strut length to least radius of gyration, will not vary in that ratio. As an approximation it may be assumed that, in passing from any grade of steel to a higher grade, if, as before, r (greater than unity) is the ratio of the elastic limits of the two metals,

$$C'_w = \frac{1}{2} C_w \left(1 + \frac{1}{r}\right) \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$\text{and } C'_c = \frac{C_c}{r} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Substituting these values in Equation 5, we have

$$T' = K + \frac{1}{r}\left(T_1 + C_c\right) + \frac{1}{2} C_w \left(1 + \frac{1}{r}\right) \ldots (8)$$

Substituting the values of K, T_1, C_c, and C_w in terms of T as previously given, we have

$$T' = T\left(0.3 + \frac{0.7}{r}\right) \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

In finding the new truss weight per linear foot for a higher steel, after computing it (as just indicated) for the direct effect of increased elastic limit, it must be corrected for the indirect effect, which is the changed total load per linear foot for trusses. This correction is made thus:

Find the sum of the live load, impact load, and dead load per linear foot of span, for the known truss weight, T, and then determine approximately the corresponding sum (on the basis of an assumed final value of T'_f) for the new truss weight. Let the ratio of these sums (less than unity) be r_1, then

$$T'_f = T' \, (0.3 + 0.7\, r_1) \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

where T'_f is the final value of the truss weight. Combining Equations 9 and 10 gives

$$T'_f = T\left(0.3 + \frac{0.7}{r}\right)(0.3 + 0.7\, r_1) \ldots\ldots\ldots\ldots (11)$$

If the computed value of T'_f does not agree quite closely with its value adopted in determining the trial dead load, a new dead load is to be assumed, and the calculations are to be made afresh. The second attempt, in all probability, will give a sufficiently accurate agreement.

On Piers.—To find the new value, P', from the old value of P, the span length being unchanged, the following approximately correct equation may be used:

$$P' = P\left(0.6 + 0.4\,\frac{r_1}{r}\right) \dots\dots\dots\dots (12)$$

where r and r_1, respectively, are the ratios previously indicated for elastic limits and total loads per linear foot of span.

In extending a curve of simple truss weights of metal per linear foot of span beyond the limits of accurate computations, the following formulas may either be used directly or as a check, the character of the steel, of course, being unchanged. Assume first that the live and the dead loads per linear foot of span remain constant, and consider the effect only of longer spans and greater truss depths. Dealing first with the chords, some 85% of their weights of metal per linear foot of span vary directly as the moments of the total loads and inversely as the truss depths; but the moments vary as the squares of the span lengths, and the stresses are inversely as the truss depths. Again, the truss depths within short limits may, without serious error, be taken to vary directly as the span lengths. Such being the case, 85% of the weights per foot of the chords will vary directly as the span lengths, or

$$C' = 0.15\,C + 0.85\,C\,\frac{l'}{l} = C\left(0.15 + 0.85\,\frac{l'}{l}\right) \dots (13)$$

where C is the chord weight per foot for the shorter span, l, and C' is the corresponding weight for the longer span, l'.

Let W and W' be, respectively, the weights of metal per linear foot of span in the webs of the two spans. About 75% of these will vary directly as the averages of all the live-load and dead-load shears on the spans, and these average shears vary almost directly as the span lengths. Again, the said 75% of W and W' will vary directly as the truss depths, and, therefore, as previously assumed, once more directly as the span lengths.

Combining these ratios will give the equation:

$$W' = 0.25\,W + 0.75\,W\left(\frac{l'}{l}\right)^2 = W\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\} \dots (14)$$

but $T = C + W$,

and $$T' = C' + W' = C\left(0.15 + 0.85\,\frac{l'}{l}\right) + W\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\} . (15)$$

It is well known that in trusses with parallel chords and of economic depths the weight of the chords is equal to the weight of the web; but, in trusses with polygonal chords and having center depths less than the theoretically economic ones, as do those of all long-span bridges, the weight of the chords is much greater than that of the web. As a general average, we may assume that $C = 0.6\ T$, and $W = 0.4\ T$, hence

$$T' = 0.6\ T\left(0.15 + 0.85\frac{l'}{l}\right) + 0.4\ T\left\{0.25 + 0.75\left(\frac{l'}{l}\right)^2\right\}$$

$$= T\left\{0.19 + 0.51\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\} \ldots\ldots\ldots\ldots (16)$$

This value of T' is based on the incorrect assumption that the total loads per linear foot of span are the same for both spans under consideration, hence it requires a further modification, as follows:

$$T'_f = T'\ (0.2 + 0.8\ r_1) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

where T'_f is the final value of the weight of truss metal per linear foot of the longer span, and r_1 (in this case greater than unity) is the ratio of the total loads per linear foot.

Combining Equations 16 and 17, we have

$$T'_f = T\left\{0.19 + 0.51\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\}(0.2 + 0.8\ r_1) \ldots (18)$$

A test of this formula, on carefully computed curves of truss weights for simple spans of nickel steel from 600 to 1 000 ft. in length, shows that slightly undue prominence has been given to the invariable portion of the weights, and that the following modification of the formula will give more accurate results:

$$T'_f = T\left\{0.15 + 0.55\frac{l'}{l} + 0.3\left(\frac{l'}{l}\right)^2\right\}(0.15 + 0.85\ r_1) \ldots (19)$$

This last formula, when tested on the truss weights of simple spans from 700 to 1 000 ft. in length for an elastic limit of 90 000 lb., gave exceedingly close results; hence it is proper to adopt it as the equation for extension of all truss weights for simple spans, and, inferentially, for those of cantilever bridges; in fact, it has been tested on some of the actually computed truss weights of cantilever bridges and found to give excellent agreement.

Attention is called to the semi-rational, semi-empirical character of these reduction and extension formulas. They are, in general, the result of long personal experience in the quick computation of metal

weights for bridges; but they have been modified slightly, as hereinbefore indicated, to agree with certain checks that have been made in this investigation. As far as practicable, the formulas of Equations 11 and 19 were used for checking each other; and the results of such checks were always satisfactory. For instance, if a curve of truss weights for one class of steel were used as a basis for finding, by Equation 11, the corresponding curve for another class of steel, the latter curve would be checked by starting from any desired point (generally where the weights of actually computed bridges cease) and passing, by using Equation 19, from one span length to another, 100 or 200 ft. greater, and continuing in this manner to the superior end of the curve.

Fig. 5 gives the total weights of metal per linear foot of span in simple-truss bridges with Class R live load. A glance at it shows the great saving in weight of metal obtained by using any of the alloy steels in preference to carbon steel. The difference is most apparent between the weights for $E = 50\,000$ lb. (the nickel steel that the metal manufacturers are willing to furnish) and those for $E = 60\,000$ lb. (the nickel steel used by the writer in his series of experiments some 8 or 10 years ago, and which alloy could be readily produced by the manufacturers through the expenditure of a little extra care and without any greater expense worth mentioning). The gradual reduction in the saving of metal with the increase of elastic limit is strikingly noticeable; and the conclusion may be drawn that, unless the extremely high-alloy steels can be obtained with only moderate increase in cost, there will be no economy in using them in simple-span bridges.

Fig. 6 gives the average total weights of metal per linear foot of span for cantilever structures having main openings of various lengths. The type of these bridges is the most common one (denominated "Type A" in "Nickel Steel for Bridges"), the live loads used being Classes R and S for the floor system and Class U for the trusses, as given in "De Pontibus." The proportional dimensions of this typical, through, cantilever bridge are as follows:

A main span, l, having a suspended span of three-eighths of l, and two cantilever arms each five-sixteenths of l in length, also two anchor arms of the same length as the cantilever arms. Any reasonable variation from these proportions would not change materially the average weight of metal per linear foot of span, as given by the curves

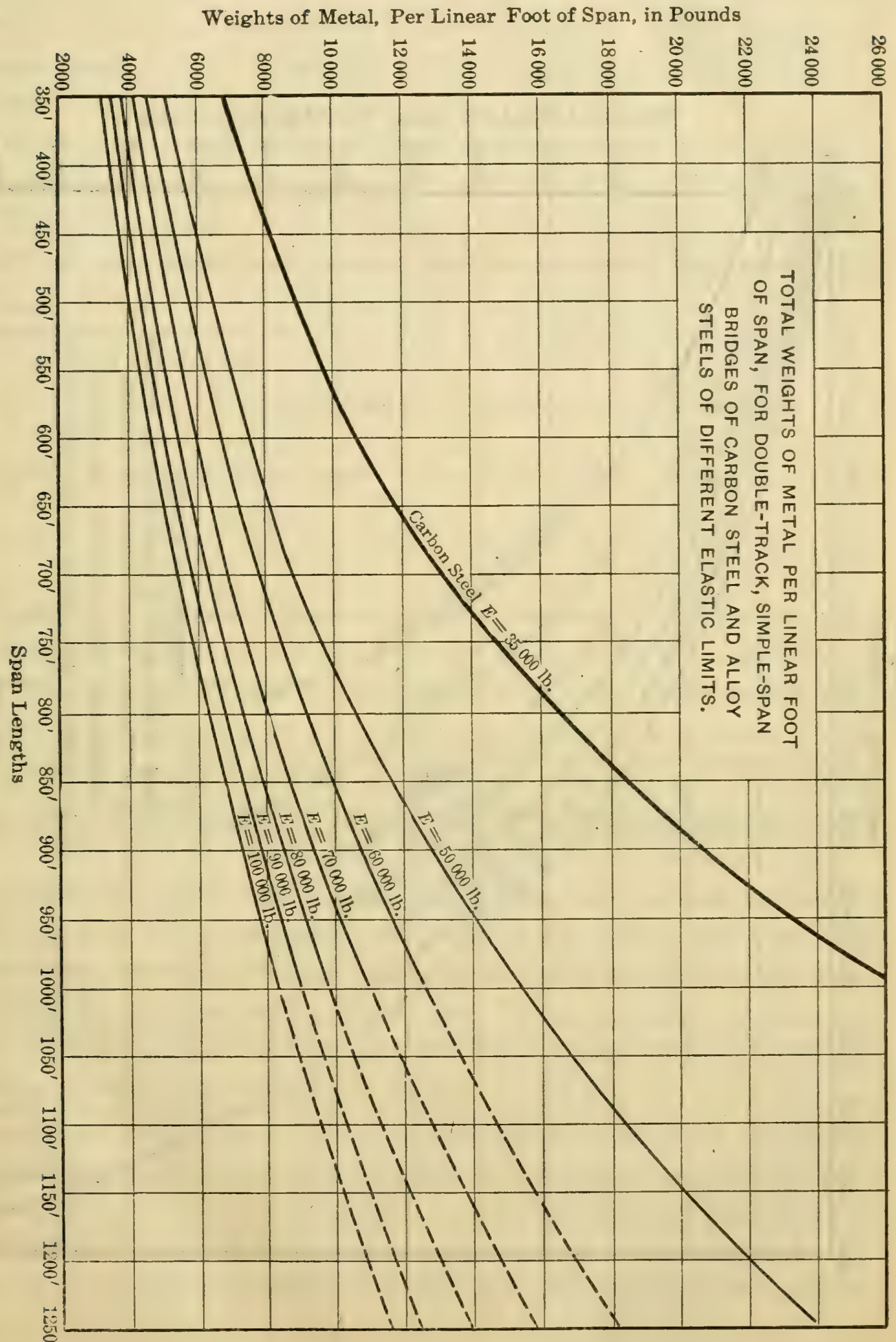

Weights of Metal, Per Linear Foot of Span, in Pounds
2000
4000
6000
8000
10 000
12 000
14 000
16 000
18 000
20 000
22 000
24 000
26 000
TOTAL WEIGHTS OF METAL PER LINEAR FOOT
OF SPAN, FOR DOUBLE-TRACK, SIMPLE-SPAN
BRIDGES OF CARBON STEEL AND ALLOY
STEELS OF DIFFERENT ELASTIC LIMITS.
Carbon Steel E = 35 000 lb.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

FIG. 5.

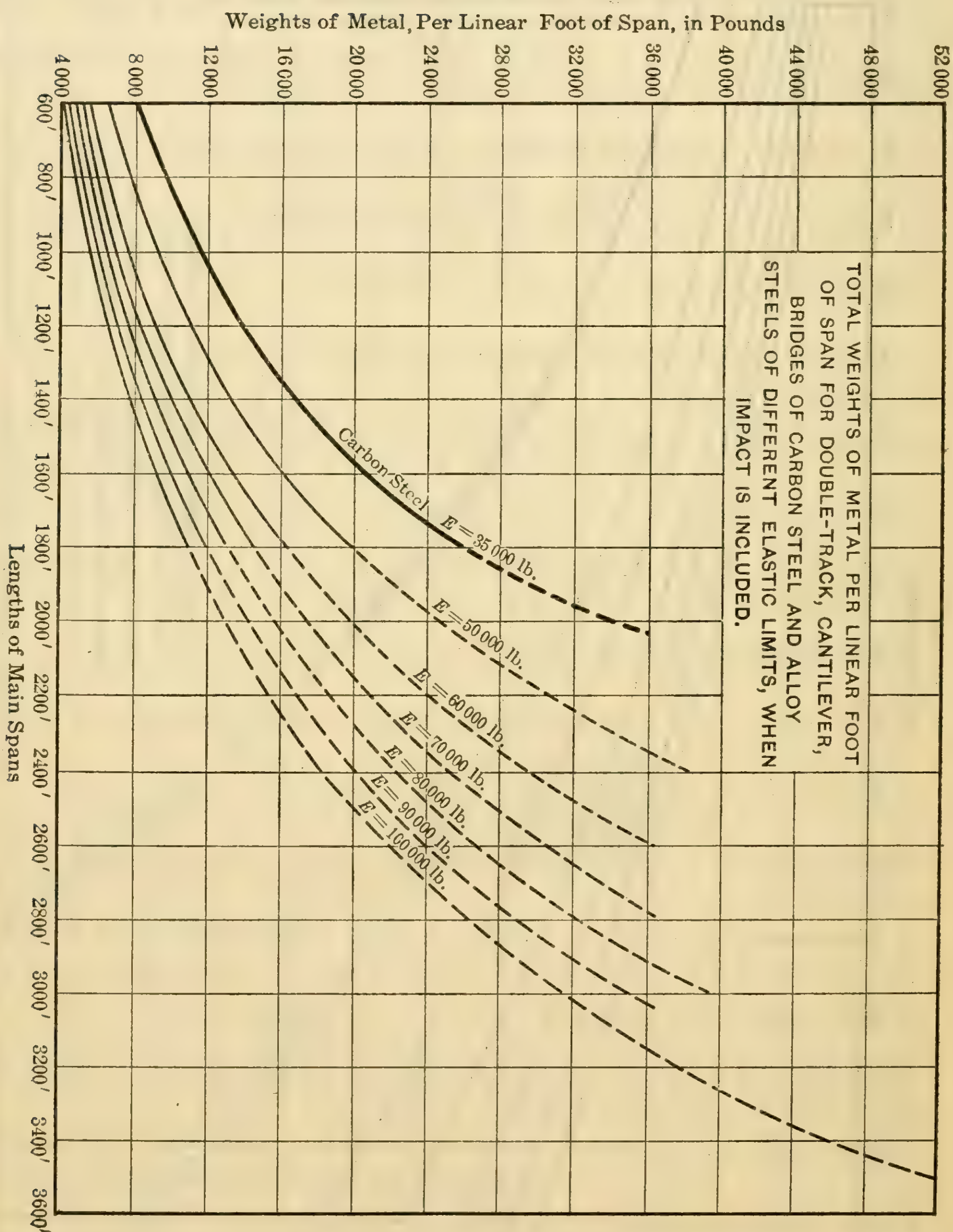

TOTAL WEIGHTS OF METAL PER LINEAR FOOT OF SPAN FOR DOUBLE-TRACK, CANTILEVER, BRIDGES OF CARBON STEEL AND ALLOY STEELS OF DIFFERENT ELASTIC LIMITS, WHEN IMPACT IS INCLUDED.
Weights of Metal, Per Linear Foot of Span, in Pounds
4 000
8 000
12 000
16 000
20 000
24 000
28 000
32 000
36 000
40 000
44 000
48 000
52 000
600′
800′
1000′
1200′
1400′
1600′
1800′
2000′
2200′
2400′
2600′
2800′
3000′
3200′
3400′
3600′
Lengths of Main Spans
Carbon Steel
E = 35 000 lb.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.

FIG. 6.

on Fig. 6. In this figure the superiority of the alloy steels over the carbon steel is just as clear as it was in the case of the diagram for simple spans, but the advantage of using very high steels is evidently greater.

If (as can be seen by the diagram to be logical) it be assumed that a limit of 36 000 lb. of metal per linear foot of span is as high as it is either economical or practicable to go in the building of double-track, railway, cantilever bridges, the corresponding limiting lengths of main openings will be approximately as follows:

For carbon steel	2 030 ft.
" $E =$ 50 000 lb. steel	2 340 "
" $E =$ 60 000 " "	2 590 "
" $E =$ 70 000 " "	2 780 "
" $E =$ 80 000 " "	2 910 "
" $E =$ 90 000 " "	3 030 "
" $E =$ 100 000 " "	3 140 "

The assumption of a limit of 36 000 lb. of metal per linear foot of span as a maximum, means that, for carbon steel, there would be required at this limit 4.35 lb. of metal to support each pound of live load that is carried (exclusive of impact allowance); and that for the alloy steels of the various elastic limits the corresponding figures are 4.37, 4.39, 4.40, 4.41, and 4.42 lb., respectively, the average of which is about 4.4 lb. From the appearance of the curves at their superior ends one may draw the conclusion that, in the case of the very high-alloy steels, the limit of weight of metal per linear foot of span can legitimately be raised beyond the said 36 000 lb. The more nearly these curves approach the vertical the more uneconomical would it be to extend the limit beyond the 36 000 lb. per lin. ft. For instance, it is plainly evident that there is no advantage in carrying the carbon-steel bridges beyond the limit of 2 000 ft. for main opening, but it is otherwise for the E = 100 000-lb. curve. Continuing this curve by deflections, it is found that the weight would reach 46 000 lb. per lin. ft. for a span of 3 400 ft.; and that the inclination from the vertical at that point is greater than that for the carbon-steel curve at its limit of 36 000 lb. with a main opening of 2 030 ft. Perhaps, therefore, in the case of the highest steel herein considered, it would be more correct to assume the extreme economic or practicable limit

of main opening to be 3 400 ft. or even 3 500 ft. For this last length the average weight of metal per linear foot of structure shown by the $E = 100\,000$-lb. curve would be 52 000 lb., which means that it would require 6.38 lb. of metal to support each pound of live load, exclusive of the effect of impact. This seems to be an excessive quantity; nevertheless, it is conceivable that conditions might exist which would render it advisable to adopt this extreme limit of main opening, although, at such a length, a suspension bridge is undoubtedly cheaper than a cantilever structure.

If it be admitted that—as is maintained by some bridge engineers—in structures of very long span the impact of the live load on the main members of trusses is essentially nil, the practicable limiting length of main opening will be somewhat increased. Moreover, such a contention is not far wrong, because the latest experiments on impact from live loads on bridges show that the effect thereof on ordinarily long spans is much less than engineers in general have been assuming during the last two decades. That the impact ever reduces actually to zero is most unlikely; but, for openings of 1 200 ft. and greater, it is true that its amount is so small as to be negligible, in view of the fact that the live-load stresses on the main truss members will never be quite as great as they are computed, because, first, the trains on the two tracks never advance together so as to produce maximum web stresses; second, such trains are not likely ever to cover entirely the bridge or even any individual part of it, except, perhaps, the central span; and, third, it is improbable that any load of cars, unless they be ore or coal cars, will ever be uniformly full and loaded to the assumed limit.

In Fig. 7 are given the curves of weights for cantilever bridges of the same type and loading as those used in preparing the curves in Fig. 6, excepting that the impact on main members of trusses is assumed to be equal to zero. The curves in Fig. 7 begin at main openings of 1 200 ft. and extend to the greatest practicable limiting lengths of such openings. A comparison of Figs. 6 and 7 shows that, by ignoring impact on trusses, there is, on the average, a saving of some 700 lb. of metal per linear foot of span, for all spans and all kinds of steel. This difference, curiously enough, is comparatively uniform for all the curves and over their entire lengths, with a few exceptions. At first thought, one might imagine that the saving should be greater

for long spans than for short ones, but it must be remembered that, as the span length increases, the percentage of live-load impact diminishes. It is this fact which makes the difference under consideration so uniform.

Under the assumption that the limit of weight of metal is 36 000 lb. per lin. ft., the greatest practicable span lengths have been

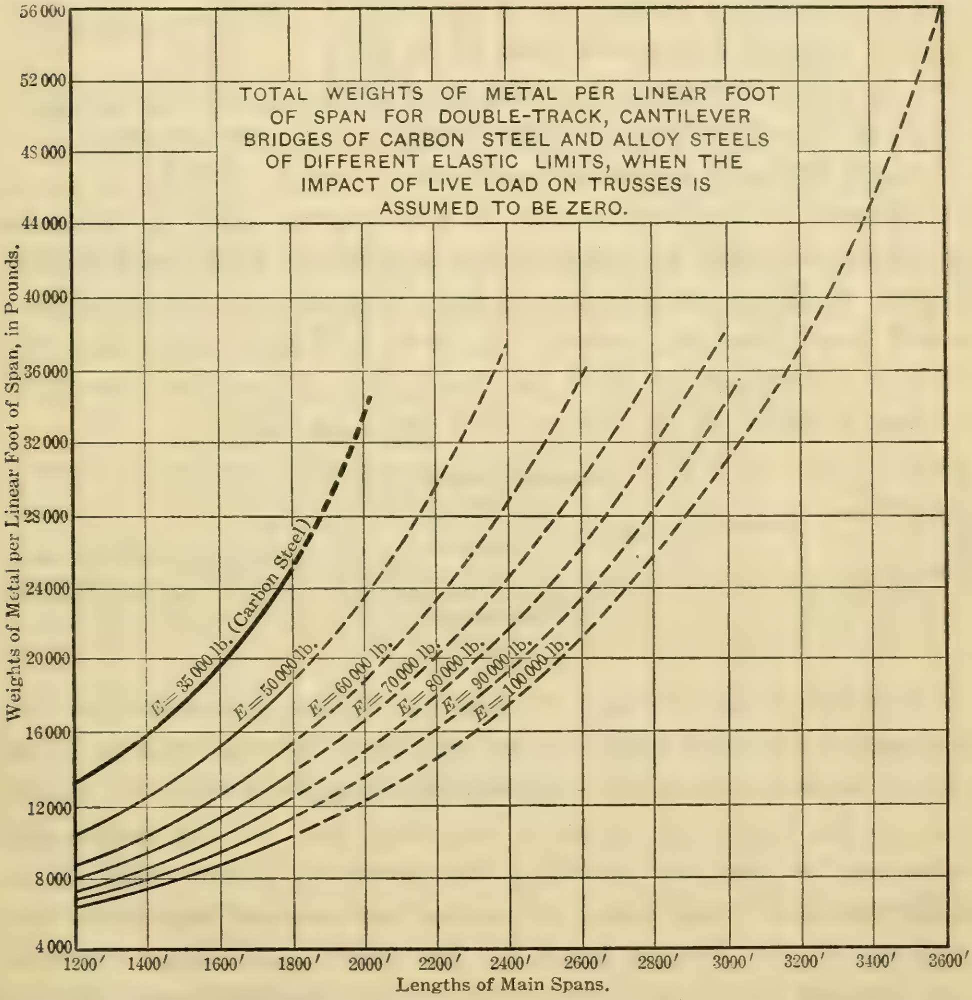


FIG. 7.

increased on the average only about 20 ft. by neglecting impact on trusses. Comparing the extensions of the curves for $E = 100\,000$ lb., it is found that, for an assumed limit of 52 000 lb. of metal per linear foot of span, the extreme practicable length of main opening has been increased only 25 ft. These various comparisons show that there is but little gain, either in economy or increase of practicable limit of opening, by neglecting the effect of impact; hence, in the economic

investigations which follow, the effect of impact has been duly considered.

In Figs. 8 and 9 are plotted, for both simple spans and cantilever bridges, the percentages of carbon steel in structures of mixed nickel and carbon steels. The curves are accurate for simple spans up to 600 ft. and for cantilever bridges up to openings of 1 800 ft., and beyond these limits they have been continued by deflections, although it is true that, in bridges built of mixed high-alloy and carbon steel, the percentages of the latter should be greater than those in the corresponding structures of mixed nickel and carbon steels, still the difference is always small; and, as the errors involved by ignoring it would be on the side of safety, the curves in Figs. 8 and 9 have been used in computing the costs per linear foot of span for the erected bridges in which all the various alloy steels are used.

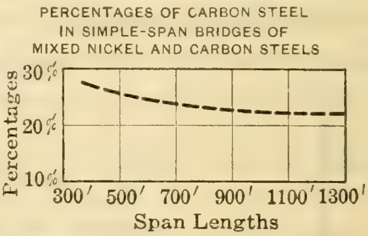


FIG. 8.

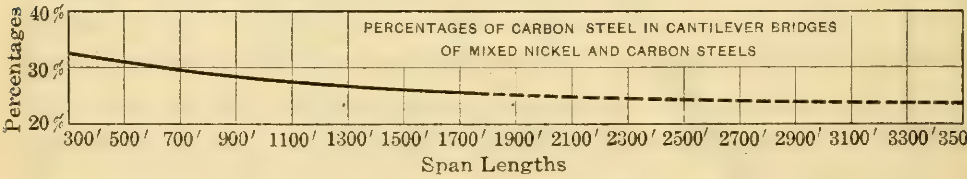


FIG. 9.

In determining the costs of erection for alloy-steel bridges, due cognizance has been taken of the fact that, for two bridges of the same span and of equal carrying capacity, though the total cost of erection in the alloy-steel bridge is less than that for the carbon-steel structure, the cost per pound in the former is greater than in the latter, because certain items of expense are constant and others vary with the weight of metal handled. The writer has assumed that one-half the total erection expense is constant and that the other half varies directly with the weight of metal. This is probably as accurate a division as can be assumed. On this basis was prepared, for the paper on "Nickel Steel for Bridges", the following mathematical statement:

Let W = weight of metal per linear foot of span in the carbon-steel bridge;

W' = ditto for the alloy-steel bridge;

C = cost per pound for erecting the carbon-steel bridge;

C' = ditto for the alloy-steel bridge;

F' = cost per linear foot for erecting the alloy-steel bridge;

then $C\ W$ = cost per linear foot for erecting the carbon-steel bridge.

$$F' = \frac{C\ W}{2}\left(1 + \frac{W'}{W}\right),$$

$$C' = \frac{F'}{W'} = \frac{C\ W}{2\ W'}\left(1 + \frac{W'}{W}\right) = \frac{C}{2}\left(\frac{W}{W'} + 1\right) \ldots\ldots (20)$$

In plotting the curves of cost in Figs. 10 to 21, inclusive, the cost per pound for the erection of the metal in the alloy-steel bridges was computed by Equation 20.

In "Nickel Steel for Bridges" it was assumed that at the time of writing (1907) the average pound prices for carbon-steel bridges erected throughout the United States were as follows:

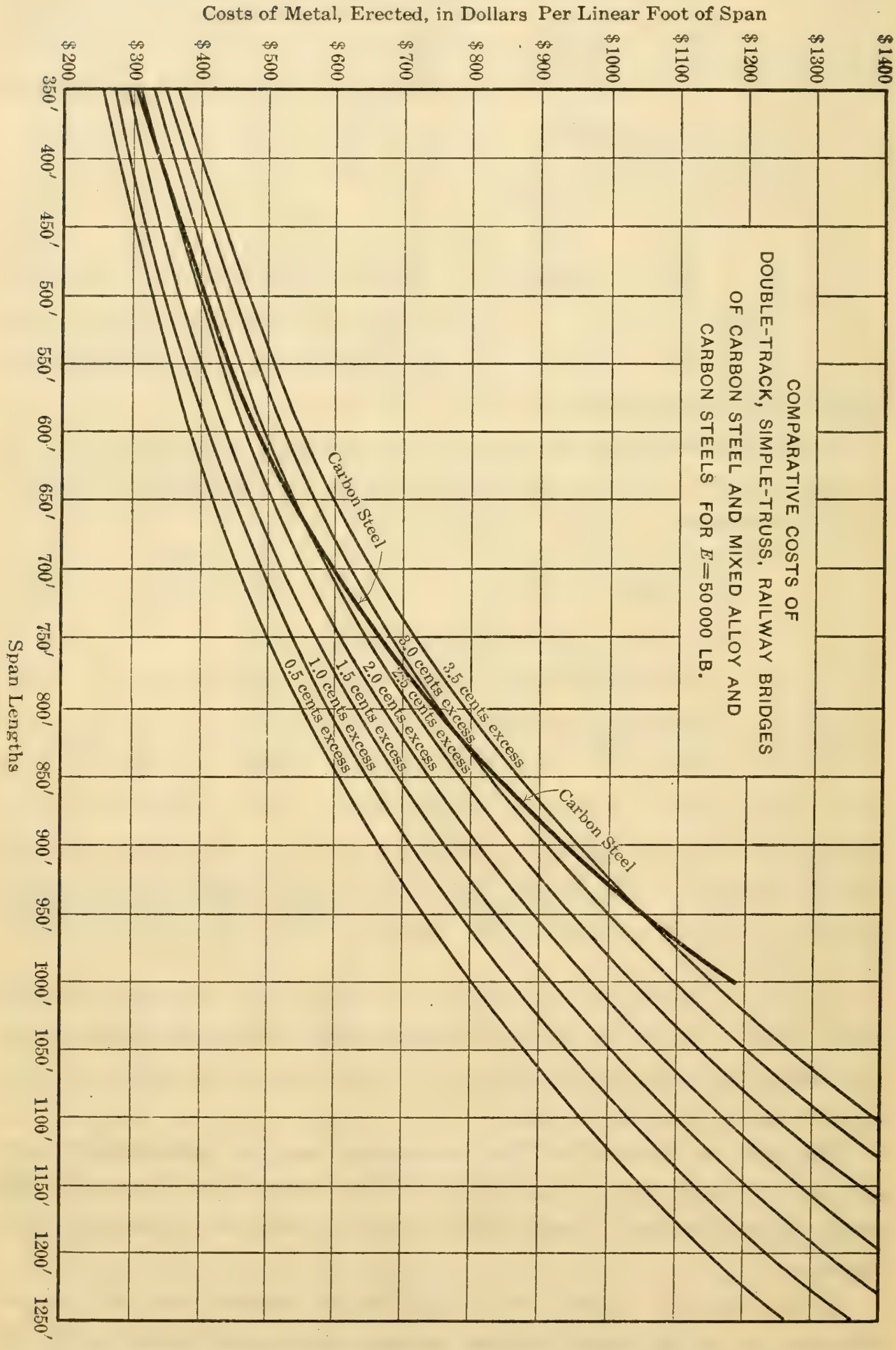
Plate-girder spans...............	4.0 cents.
Riveted-truss spans.............	4.5 "
Pin-connected, Pratt-truss spans..	4.5 "
Pin-connected, Petit-truss spans..	5.0 "
Cantilever bridges..............	5.5 "

At the present time, some 6 years later, prices are about ½ cent per lb. less; hence, for this investigation, the writer uses 4.5 cents for fixed spans and 5.0 cents for cantilever bridges. The reason for the greater assumed pound cost of the latter is mainly expensive erection, because cantilevers are generally adopted where the erection conditions are costly.

In carbon-steel bridges the proportion of the total cost of the erected metal which pertains to the erection has been arbitrarily assumed for convenience at 20%, which figure will come very close to the average for any large number of cases.

The cost per linear foot for the tracks has, for convenience, been omitted in computing all curves, as it is constant and common to all spans; but, of course, their weight was included in estimating dead loads.

As no one can foretell what will be the excess costs per pound, delivered at bridge site, for the various alloy-steel bridges over the corresponding costs for carbon-steel bridges, in this investigation



Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=50000 LB.
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Carbon Steel
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

Fig. 10.

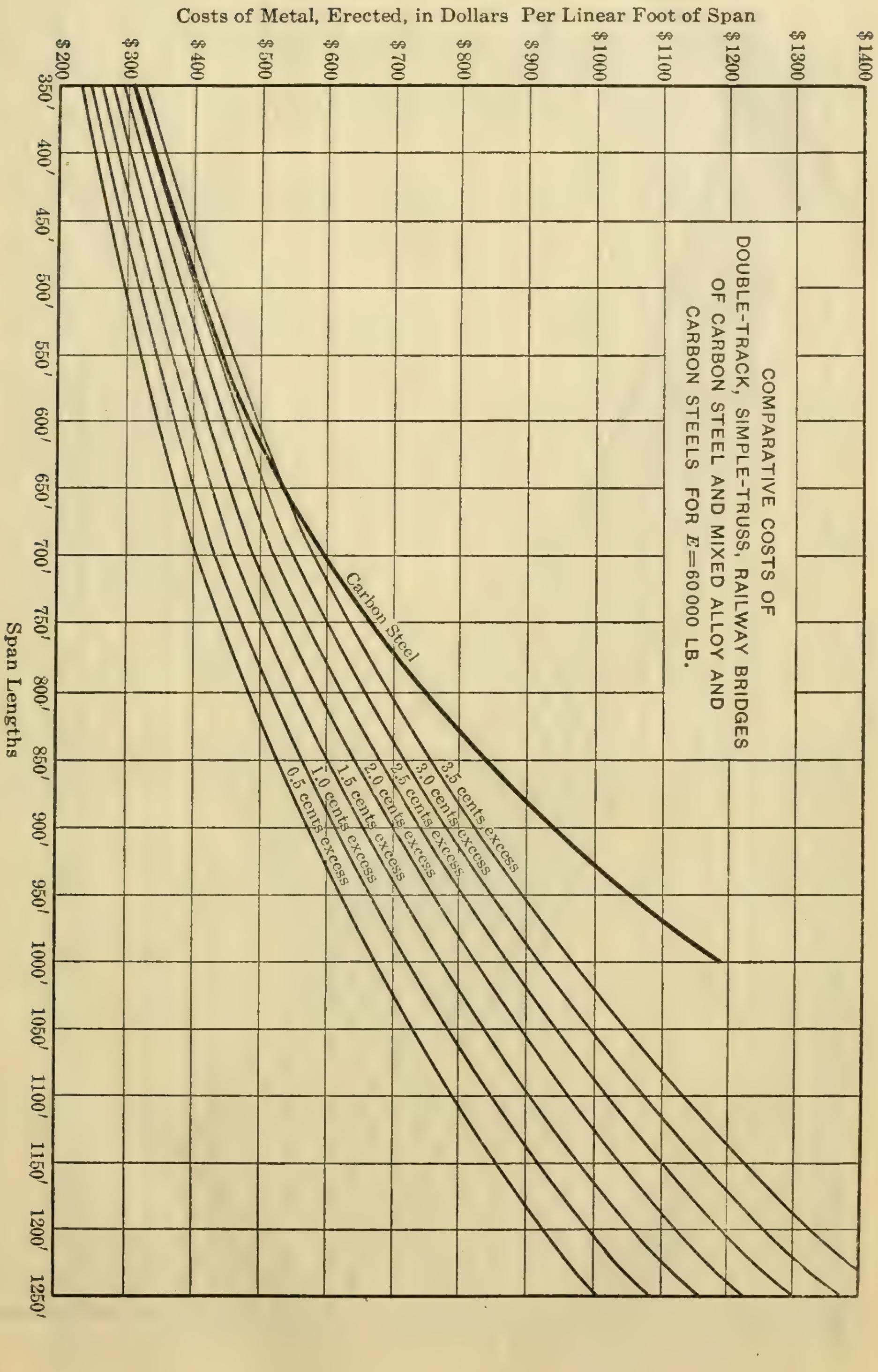

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=60000 LB.
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

FIG. 11.

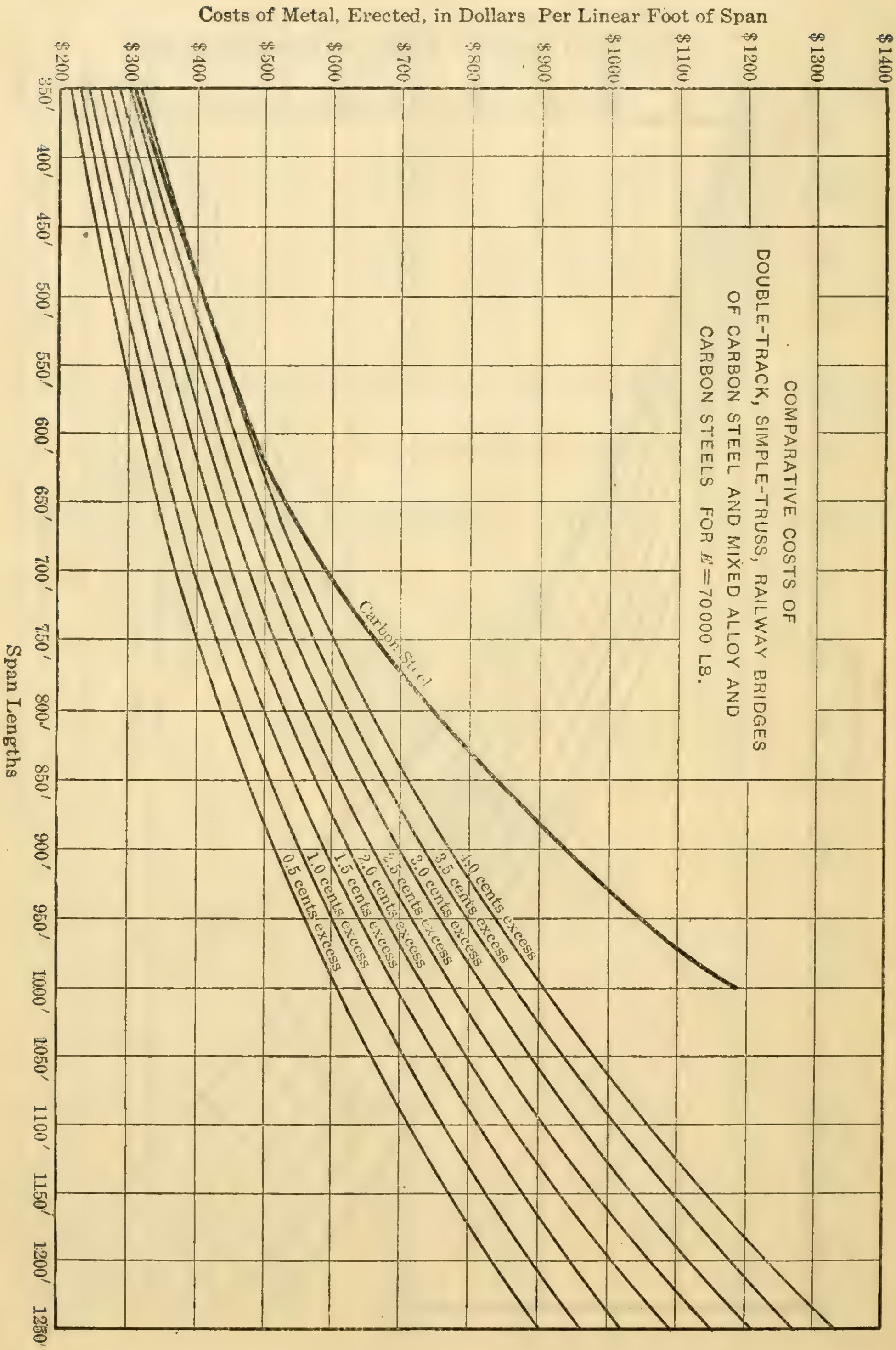

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E = 70 000 LB.
Carbon Steel
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

Fig. 12.

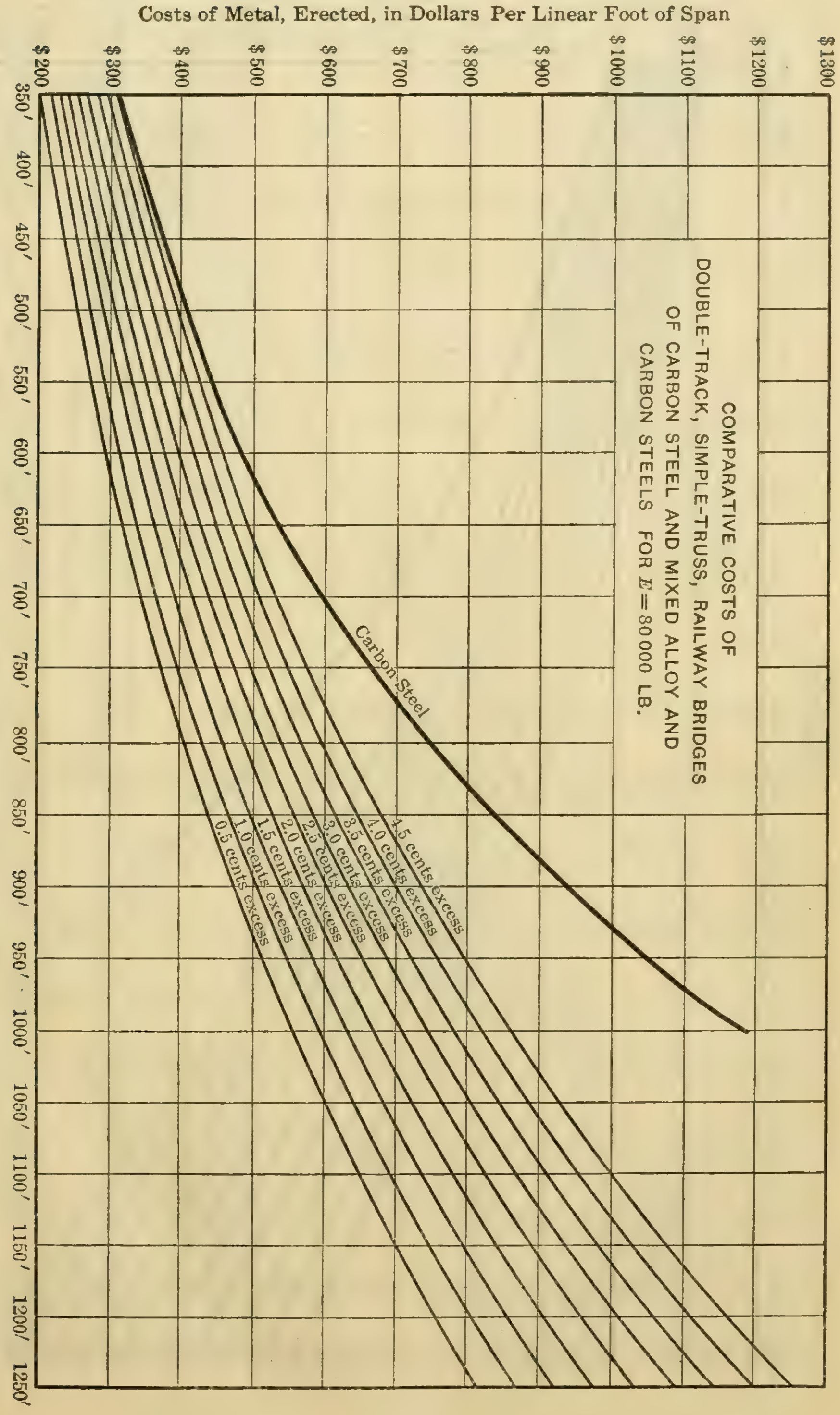

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=80000 LB.
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

Fig. 13.

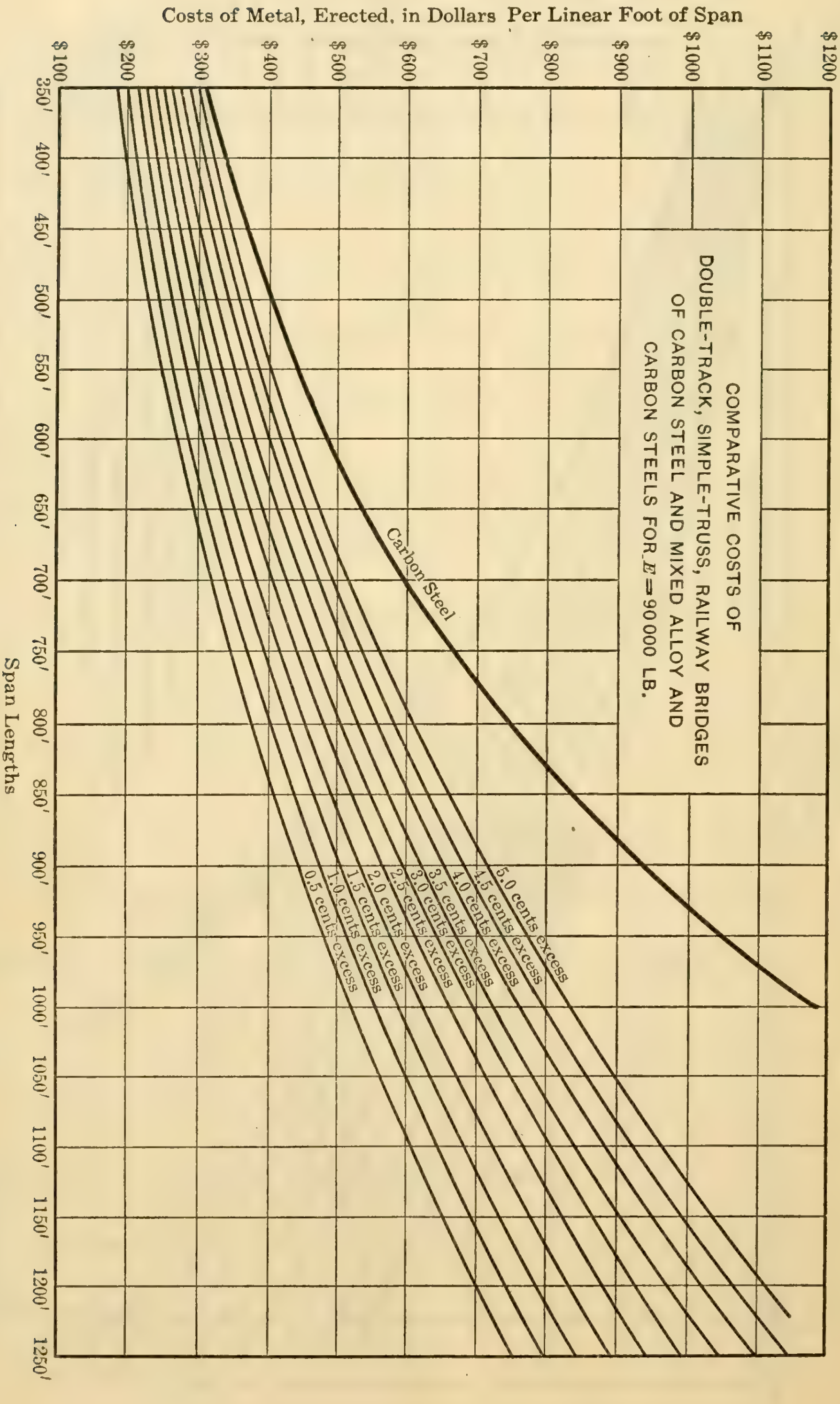

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E=90000 LB.
Carbon Steel
5.0 cents excess
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

FIG. 14.

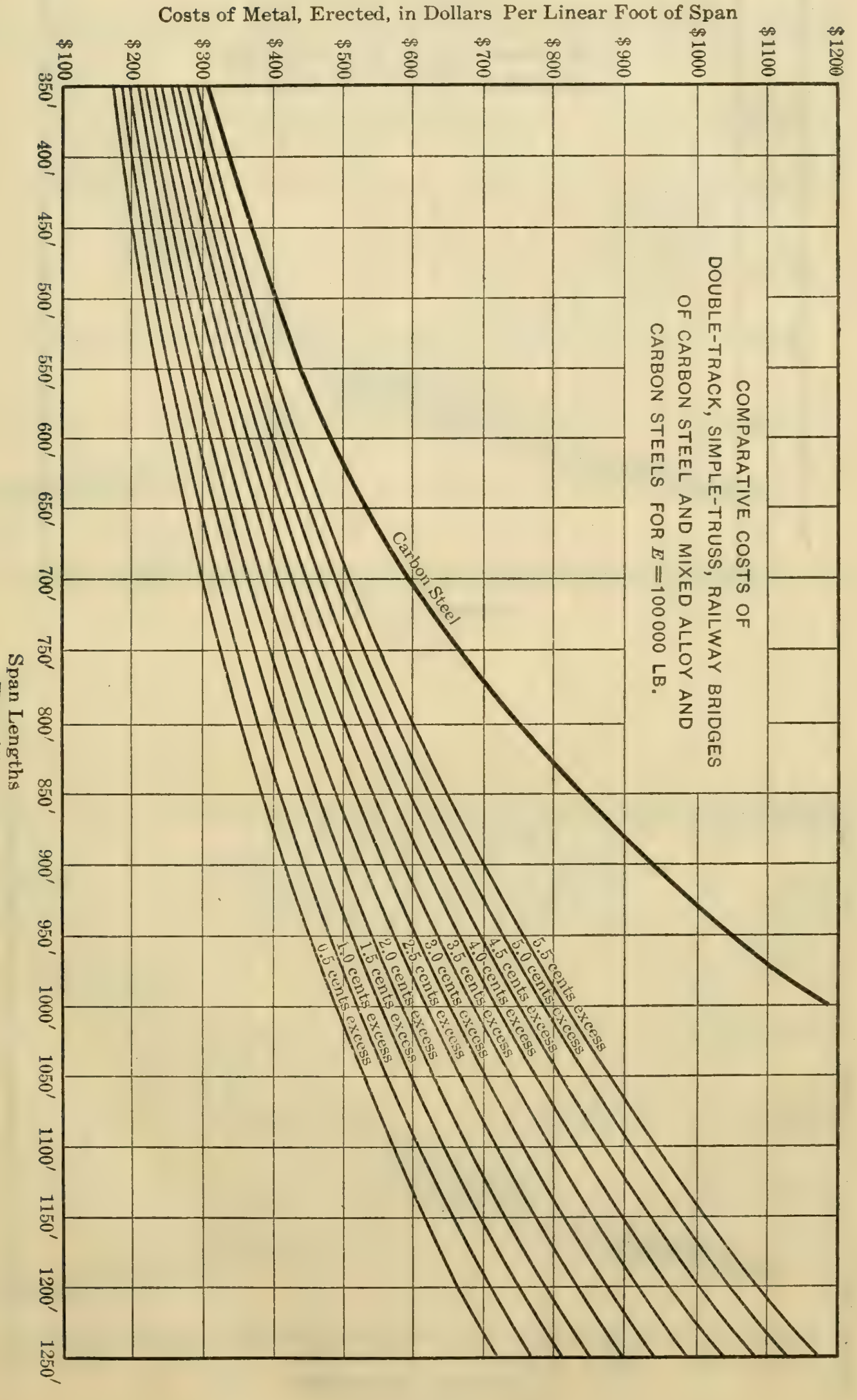

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
COMPARATIVE COSTS OF
DOUBLE-TRACK, SIMPLE-TRUSS, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY AND
CARBON STEELS FOR E = 100 000 LB.
Carbon Steel
5.5 cents excess
5.0 cents excess
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

FIG. 15.

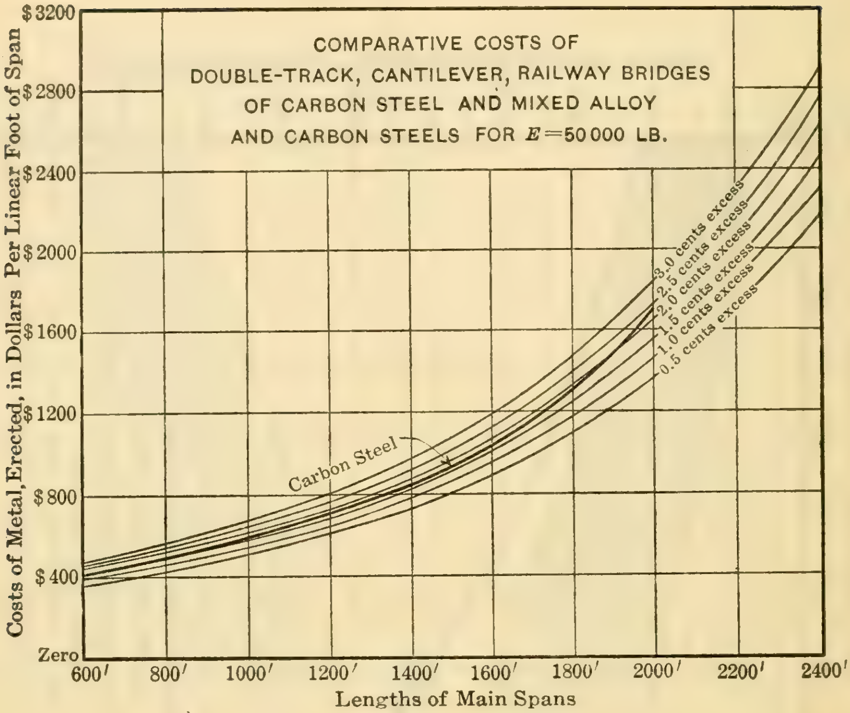

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=50 000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′
800′
1000′
1200′
1400′
1600′
1800′
2000′
2200′
2400′
Lengths of Main Spans
Carbon Steel
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 16.

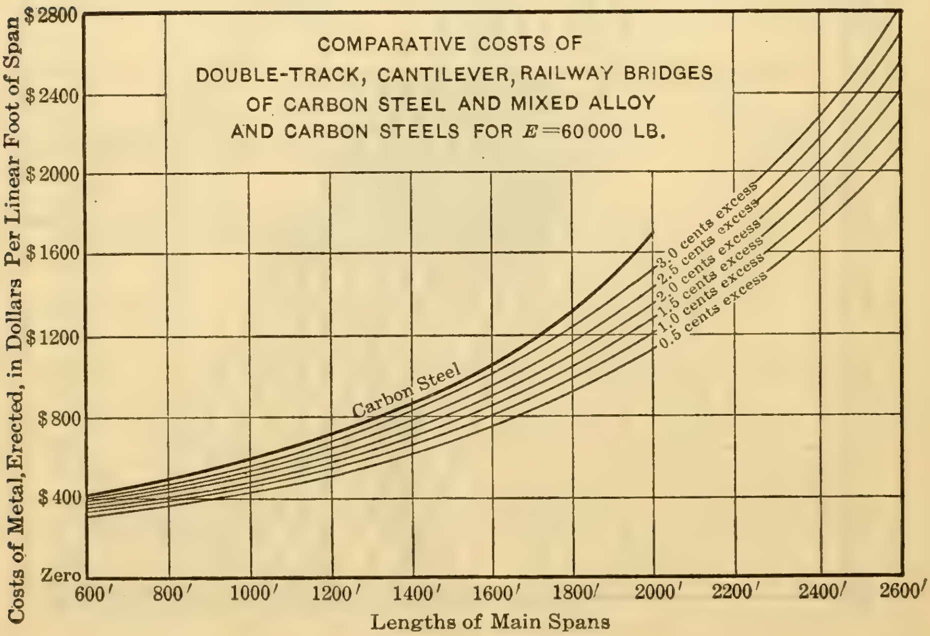

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR E=60 000 LB.
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′
800′
1000′
1200′
1400′
1600′
1800′
2000′
2200′
2400′
2600′
Lengths of Main Spans
Carbon Steel
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess

FIG. 17.

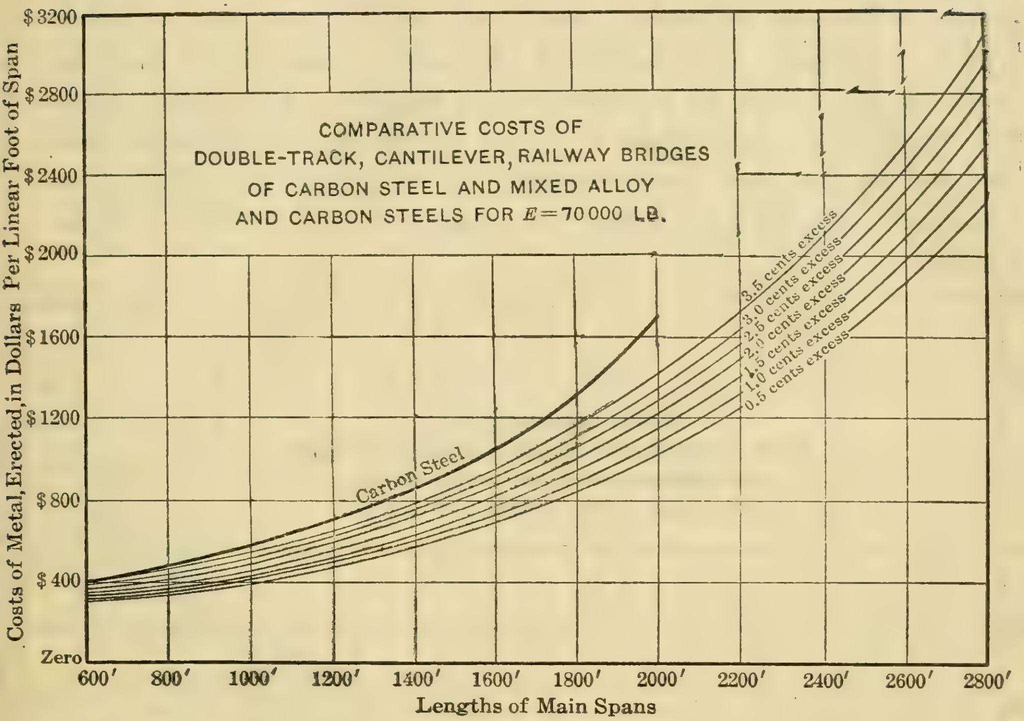

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR $E=70\,000$ LB.
Carbon Steel
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′
Lengths of Main Spans

FIG. 18.

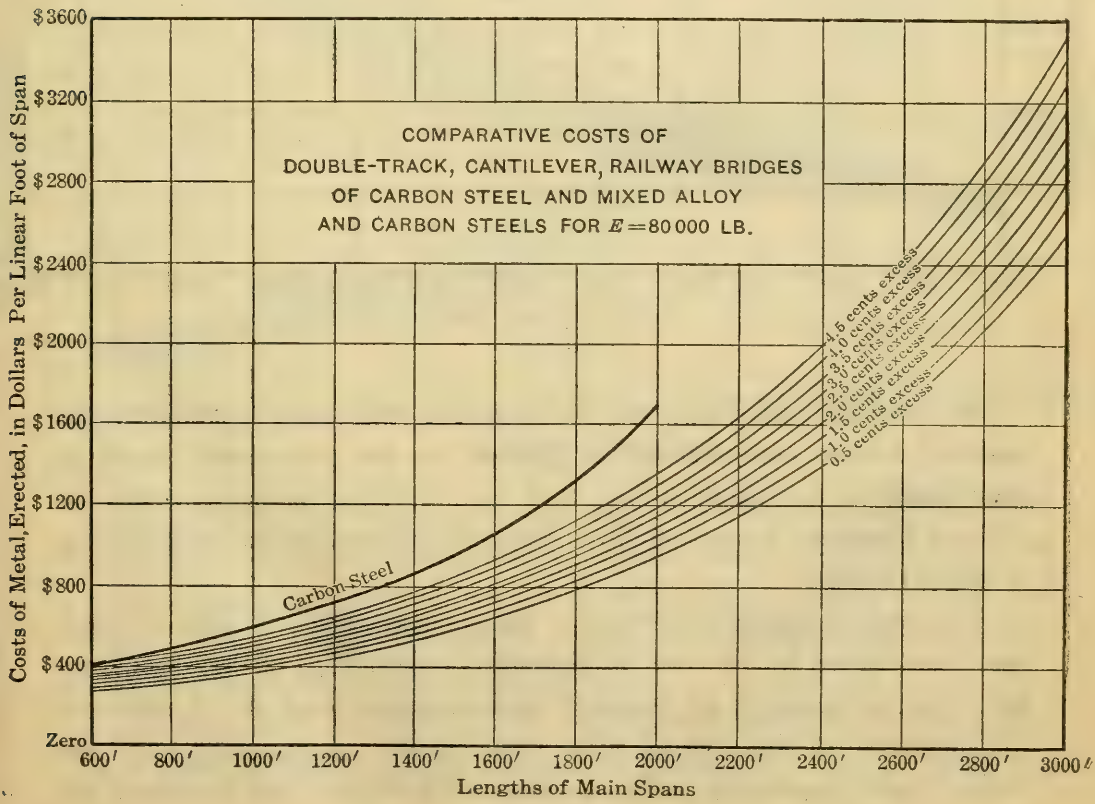

COMPARATIVE COSTS OF
DOUBLE-TRACK, CANTILEVER, RAILWAY BRIDGES
OF CARBON STEEL AND MIXED ALLOY
AND CARBON STEELS FOR $E=80\,000$ LB.
Carbon Steel
4.5 cents excess
4.0 cents excess
3.5 cents excess
3.0 cents excess
2.5 cents excess
2.0 cents excess
1.5 cents excess
1.0 cents excess
0.5 cents excess
Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$3600
$3200
$2800
$2400
$2000
$1600
$1200
$800
$400
Zero
600′ 800′ 1000′ 1200′ 1400′ 1600′ 1800′ 2000′ 2200′ 2400′ 2600′ 2800′ 3000′
Lengths of Main Spans

FIG. 19.

various differences, beginning with ½ cent per lb., have been assumed; so that, for bridges built mainly of alloy-steel, of any elastic limit up to 100 000 lb., and for all practicable span lengths, in both simple spans and cantilevers, a comparison of cost of erected structure between that steel and all the other steel considered for all reasonable variations in pound price can be made either at a glance or very quickly by interpolation.

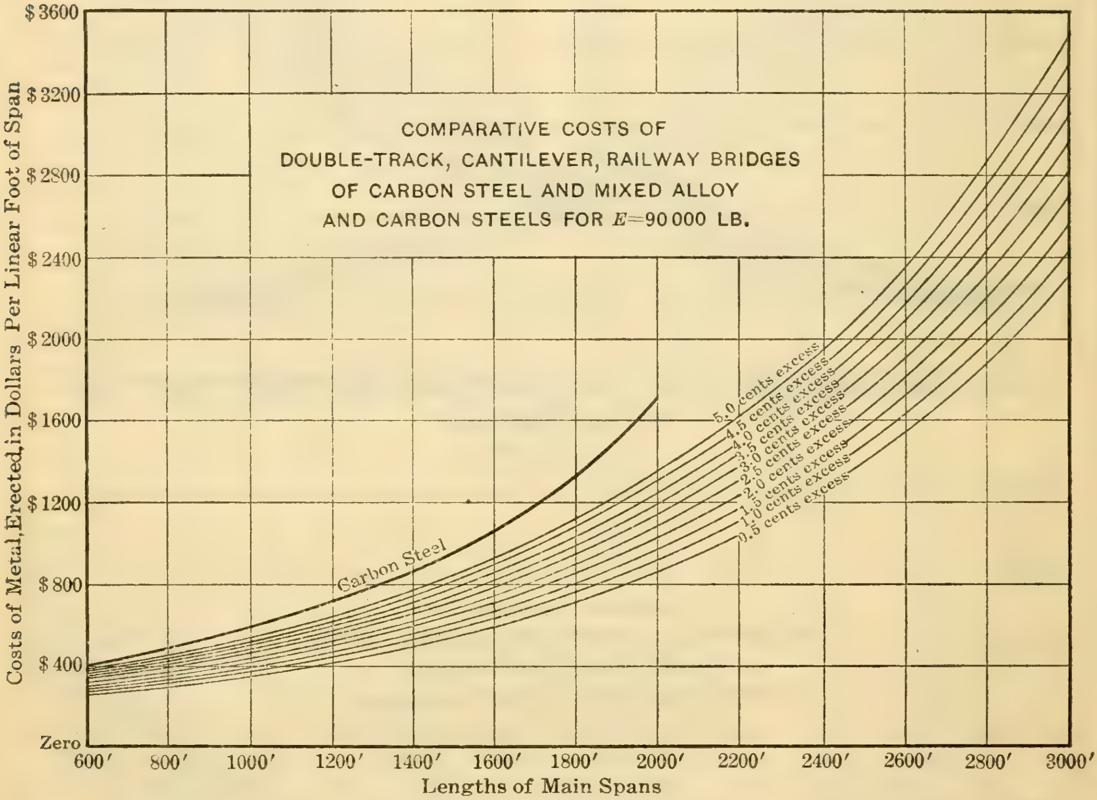


FIG. 20.

An inspection of Figs. 10 to 21, inclusive, will show how rapidly the costs of bridge superstructures increase as the practicable limit of span length is approached; also, that for very long spans an additional price of ½ cent or 1 cent per lb. is of small importance in the total cost of the structure.

In order to make evident at a glance the effect on cost of long-span cantilevers by the use of high-alloy steels, Fig. 22 has been prepared, on the basis of an assumed uniform excess cost of 2.5 cents per lb., delivered at site, for all alloy steels above the standard price for carbon steel, beginning with openings of 1 200 ft. and carrying the

curves out to the practicable limits of construction. This diagram indicates conclusively the folly of using nickel steel of an elastic limit of 50 000 lb., when, for practically the same price, an almost identical alloy having an elastic limit of 60 000 lb. is obtainable. Again, this diagram shows, for the different steels, the main openings for cantilever bridges, which, other things being equal, involve the same cost of structure per linear foot of span; it also shows how the differ-

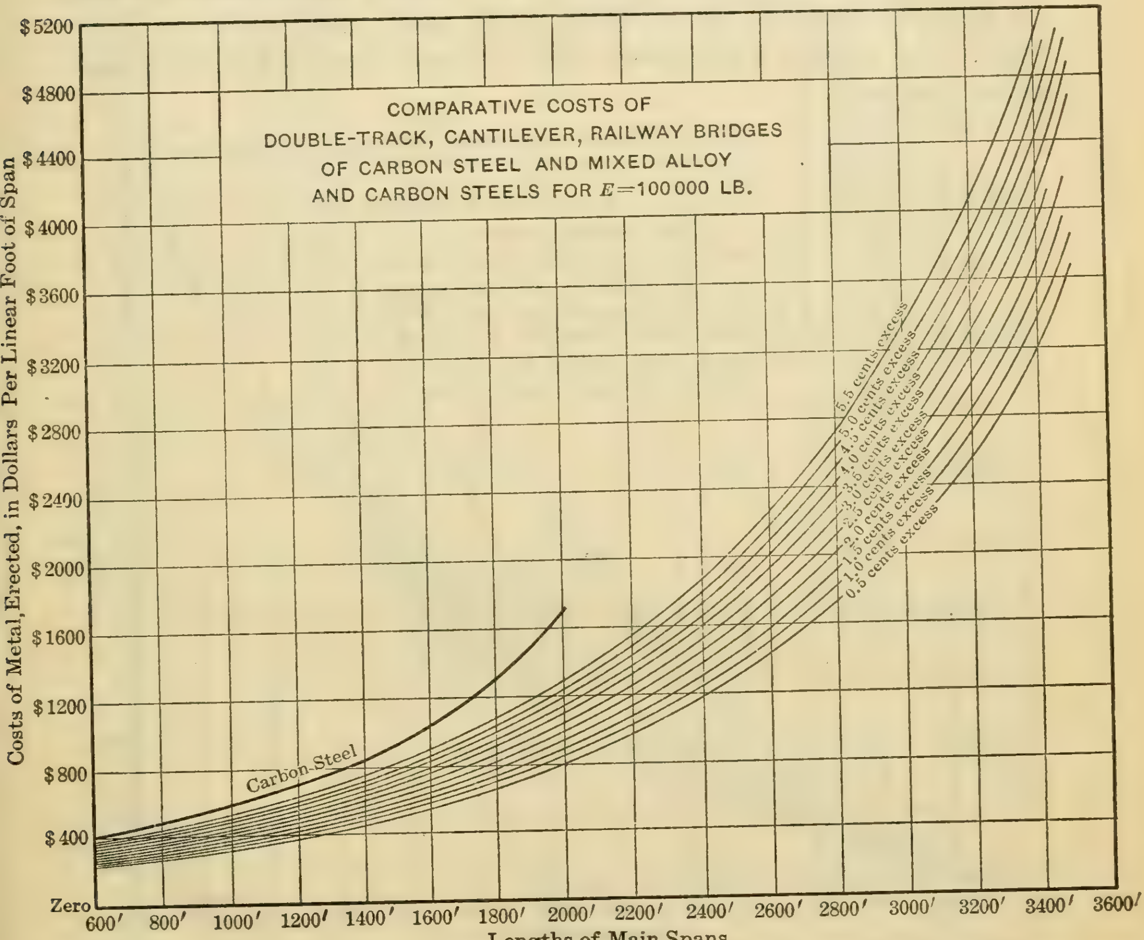


FIG. 21.

ences between the lengths of such main openings decrease regularly with the increase of elastic limit of the metal. Finally, the great upward tendency of the curve for $E = 100\,000$ lb. near its outer end shows that, for such an elastic limit, the extreme practicable span length is reached at about 3 500 ft.

In Fig. 23 is given the same information concerning simple-truss spans that is furnished for cantilevers by Fig. 22, *viz.*, a comparison

of costs of metal per linear foot of span for all the alloy steels, on the basis of adopting a uniform excess price over carbon steel of 2.5 cents per lb.

Incidentally, Figs. 22 and 23 afford an excellent check on the correctness of the writer's numerical computations; for, if there were any ordinary error made, it would be indicated at once by irregularity, either in the curves or their spacing; in fact, the plotting of these two diagrams detected the existence of two small errors at the inferior ends of the curves, which errors had not been noticeable on the preceding diagrams.

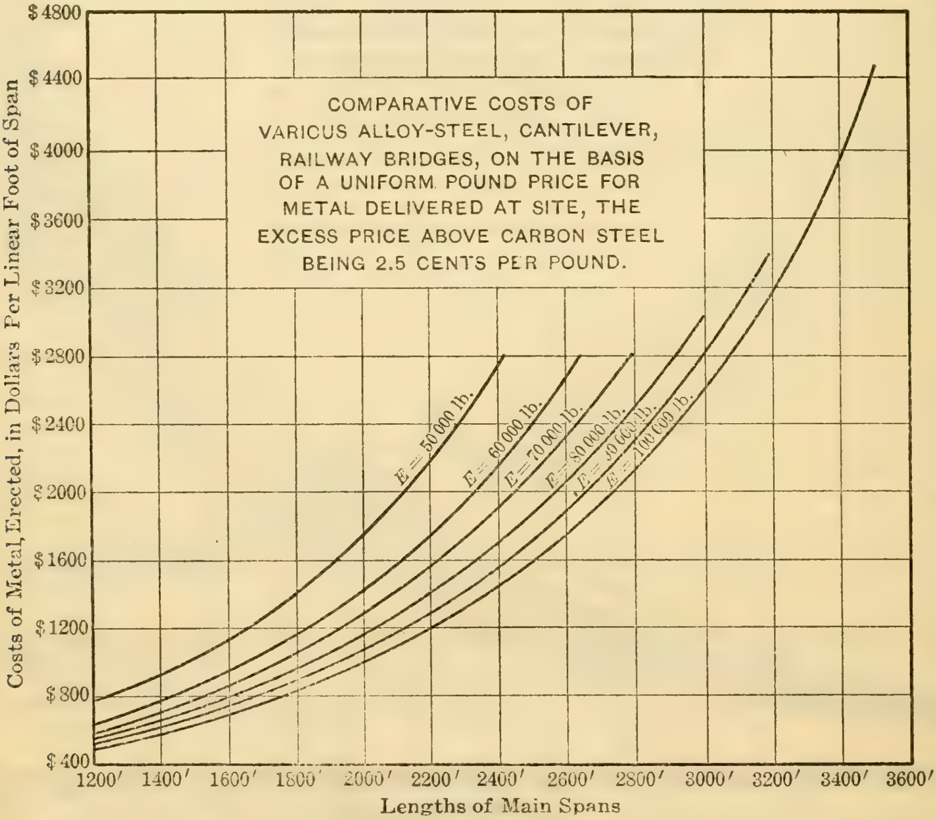


FIG. 22.

This brings up the question of the correctness of all the work in this investigation. There can exist nowhere any small errors of any importance, because they would have been detected at once by the lack of continuity in the diagrams. This feature, though, would not prevent the existence of fundamental errors based on incorrect assumptions or on wrong primary data. Such fundamental errors, however, are really impossible, because the weights of carbon-steel bridges dia-

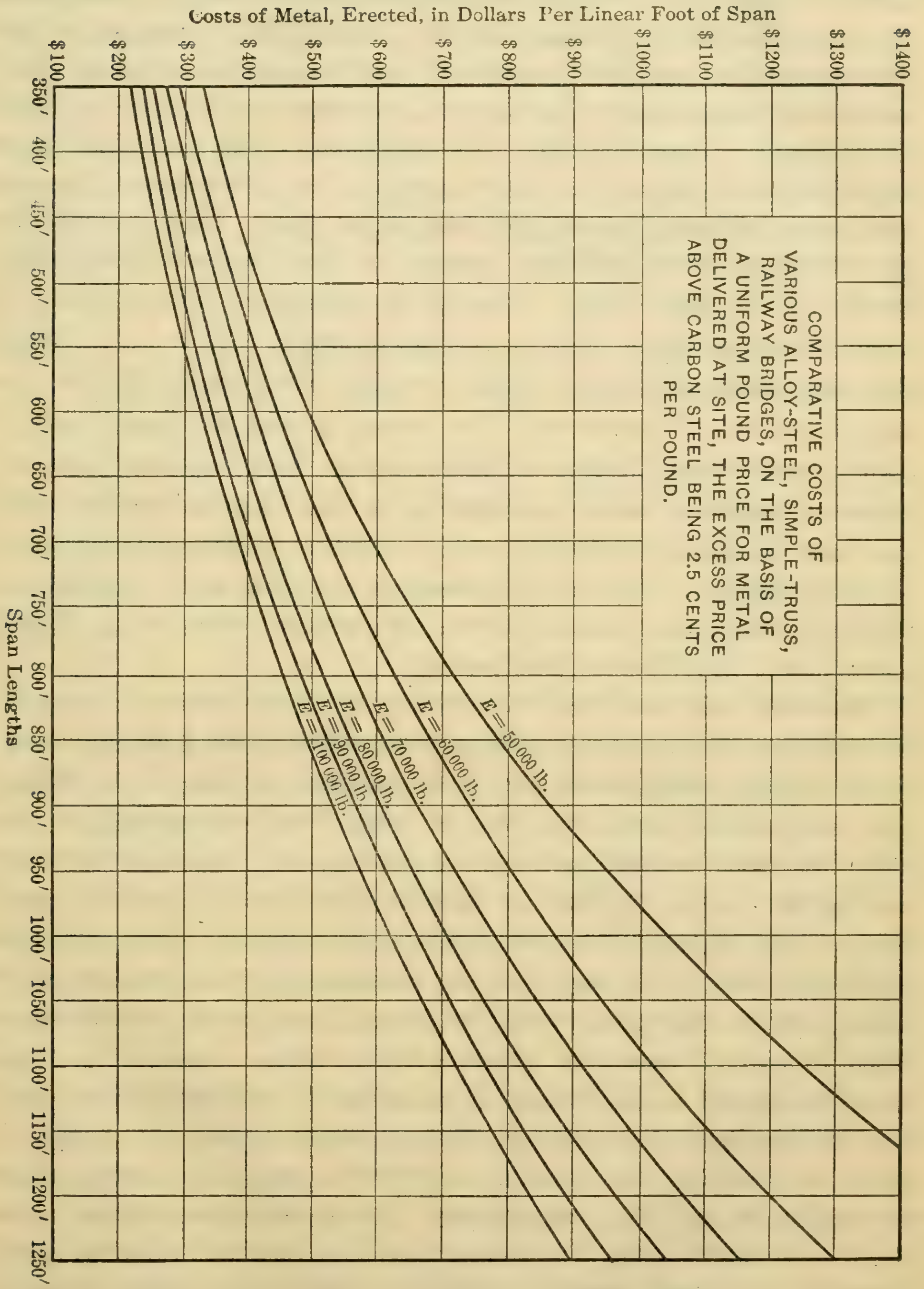

Costs of Metal, Erected, in Dollars Per Linear Foot of Span
$100
$200
$300
$400
$500
$600
$700
$800
$900
$1000
$1100
$1200
$1300
$1400
COMPARATIVE COSTS OF VARIOUS ALLOY-STEEL, SIMPLE-TRUSS, RAILWAY BRIDGES, ON THE BASIS OF A UNIFORM POUND PRICE FOR METAL DELIVERED AT SITE, THE EXCESS PRICE ABOVE CARBON STEEL BEING 2.5 CENTS PER POUND.
E = 50 000 lb.
E = 60 000 lb.
E = 70 000 lb.
E = 80 000 lb.
E = 90 000 lb.
E = 100 000 lb.
350′
400′
450′
500′
550′
600′
650′
700′
750′
800′
850′
900′
950′
1000′
1050′
1100′
1150′
1200′
1250′
Span Lengths

FIG. 23.

grammed in the writer's office are very exact. Of course, the personal equation of the designer affects the weight of metal in structures designed by him, but this variation is confined to small limits. Again, the specifications used in the designing affect materially the weights, but the diagrams under consideration were all based on a single standard specification for carbon-steel bridges and on a similar specification for nickel-steel bridges. About the only place where the writer could have gone astray in this work is in the determination of the percentages of carbon steel in the bridges of mixed alloy and carbon steels. These percentages are subject to great variation, because no two designers would agree exactly as to what minor parts of an alloy-steel bridge should be made of carbon steel. This is a question which affects only slightly the diagrammed weights of metal, but does affect materially the diagrammed costs of structures. The curves giving the percentages of carbon steel in such bridges, as before stated, were prepared from diagrams of weights of metal in bridges of mixed nickel and carbon steels; and these diagrams were the result of careful, detailed computations of actual designs. The writer, consequently, feels truly confident in regard to the correctness of all the information which he is offering herein to the Engineering Profession.

Reference was made near the beginning of this paper to a method of producing nickel steel by adding ferro-nickel instead of pure nickel to the molten carbon steel, and to a statement by Mr. T. L. Willson that ferro-nickel containing 10% of nickel and nearly 90% of iron could be manufactured and sold at a profit at 2 cents per lb., thus making the nickel content cost only 10 cents per lb., and the excess price of the ingot nickel steel about 0.3 cent per lb. The extra cost of the shopwork is 0.15 cent per lb.; consequently the total cost of the manufactured alloy metal would be about ½ cent per lb. Referring to Figs. 16 and 17, using the lowest of the curves in each, it is found that there would be an economy in using the $E = 50\,000$-lb. steel varying for cantilever bridges from 13% in spans of 600 ft. to 20% in spans of 2 000 ft. For the $E = 60\,000$-lb. steel, the corresponding figures are 24 and 33% respectively. For simple-truss bridges, using the $E = 50\,000$-lb. steel, it is found from Fig. 10, that the saving varies from 16% for 350-ft. spans, to 32% for 1 000-ft. spans; and, using the $E = 60\,000$-lb. steel and Fig. 11, the corresponding savings are 26 and 44%, respectively. From the preceding figures of economy it is evi-

dent that it would pay the builders of large bridges to experiment with the use of ferro-nickel in the manufacture of nickel steel for bridge building, so as to determine whether the claims made by Mr. Willson in regard to its use are borne out by the facts.

The weights of metal for double-track railway bridges given in Figs. 5 and 6 can be utilized in estimating costs of any long-span railway bridges, because, if there be more than two tracks, the weights of metal per linear foot of span will, *ceteris paribus*, be directly proportional to the number of tracks. This is because, if only two trusses be used, the small saving in metal, in the trusses and the lateral system, will be offset by the extra weight of the floor-beams; and, if more than two trusees be adopted (as would generally be the case so as to avoid truss members of excessive cross-section), the economy of metal would be but slight.

Should a different live load per track from those herein used be desired, the weight curves on the diagrams can be modified accordingly by using the formulas in Equations 3, 4, 10, and 12; but, to do this correctly, one would need to know the division of total weights of metal per linear foot of span between the four components, "Floor System", "Lateral System", "Trusses", and "On Piers." Although it is not practicable, on account of space restriction, to give in this paper such a division with great accuracy, Tables 1 to 6, inclusive, will enable any one to calculate approximately, for any length of span and any kind of bridge herein included, the division of metal required.

However useful, though, may be the information given concerning weights of metal per linear foot of span for bridges in general, the principal object of this paper is to indicate the possibilities in bridge construction that may be attained by the use of high-alloy steels, and it is evident that the results of the writer's computations clearly prove that a systematic series of experiments made in search of a suitable and satisfactory alloy would be well worth while. Already it is practicable to obtain plate and shape nickel steel of 60 000 lb. elastic limit and eye-bar nickel steel of 65 000 lb. elastic limit; and, in the writer's opinion, it would not take much experimenting to raise each of these figures 10 000 lb.; but, to attain an elastic limit of 100 000 lb. or even 90 000 lb. for an alloy steel suitable for all shop manipulations is truly a great problem, and one worthy of much effort and a large expenditure of time and money.

TABLE 1.—Floor System for Simple Spans.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.		
	For 350-ft. span.	For 600-ft. span.	For 1 000-ft. span.
Carbon steel	1 400	1 550	2 000
$E =$ 50 000 lb.	1 150	1 300	1 750
$E =$ 60 000 "	1 000	1 150	1 600
$E =$ 70 000 "	900	1 050	1 500
$E =$ 80 000 "	850	1 000	1 400
$E =$ 90 000 "	800	950	1 300
$E =$ 100 000 "	750	900	1 200

TABLE 2.—Lateral System for Simple Spans.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.		
	For 350-ft. span.	For 600-ft. span.	For 1 000-ft. span.
Carbon steel	450	600	1 200
$E =$ 50 000 lb.	450	600	1 150
$E =$ 60 000 "	450	600	1 100
$E =$ 70 000 "	450	600	1 050
$E =$ 80 000 "	450	600	1 000
$E =$ 90 000 "	450	600	950
$E =$ 100 000 "	450	600	900

TABLE 3.—On Piers for Simple Spans.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.	
	For 350-ft. span.	For 1 000-ft. span.
Carbon steel	250	400
$E =$ 50 000 lb	200	300
$E =$ 60 000 "	190	280
$E =$ 70 000 "	180	260
$E =$ 80 000 "	170	240
$E =$ 90 000 "	160	220
$E =$ 100 000 "	150	200

The writer is of the opinion that the first step to take is to experiment on "purified" steel, so as to bring it to its maximum of effectiveness, then to try adding nickel in various quantities, and afterward nickel and other but cheaper substances. Of course, augmenting the quantity of carbon in the purified steel, while increasing both its

TABLE 4.—FLOOR SYSTEM FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800-ft. span.	2 400-ft. span.	3 000-ft. span.	3 600 ft. span.
Carbon steel.........	1 600	1 800	2 000	....	....	....
$E =$ 50 000 lb........	1 200	1 450	1 650	2 400	....	....
$E =$ 60 000 "	1 000	1 200	1 400	2 100	....	....
$E =$ 70 000 "	950	1 150	1 300	1 850	....	....
$E =$ 80 000 "	900	1 050	1 200	1 700	2 200	....
$E =$ 90 000 "	850	950	1 100	1 600	2 000	....
$E =$ 100 000 "	800	900	1 000	1 500	1 900	2 400

TABLE 5.—LATERAL SYSTEM FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800-ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel.........	800	1 100	1 400	....	....	....
$E =$ 50 000 lb........	800	1 050	1 300	1 800	....	....
$E =$ 60 000 "	800	1 000	1 200	1 600	....	....
$E =$ 70 000 "	800	1 000	1 200	1 600	2 000	....
$E =$ 80 000 "	800	1 000	1 200	1 600	2 000	....
$E =$ 90 000 "	800	1 000	1 200	1 600	2 000	2 400
$E =$ 100 000 "	800	1 000	1 200	1 600	2 000	2 400

TABLE 6.—ON PIERS FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weights of Metal per Linear Foot of Span, in Pounds.					
	600-ft. span.	1 200-ft. span.	1 800 ft. span.	2 400-ft. span.	3 000-ft. span.	3 600-ft. span.
Carbon steel........	700	1 100	2 100	....	....	....
$E =$ 50 000 lb	600	900	1 700	....	....	....
$E =$ 60 000 "	580	860	1 600	2 200	....	....
$E =$ 70 000 "	560	820	1 500	2 100	3 500	....
$E =$ 80 000 "	540	780	1 400	2 000	3 200	....
$E =$ 90 000 "	520	740	1 300	1 900	2 900	3 900
$E =$ 100 000 "	500	700	1 200	1 800	2 600	3 600

ultimate strength and its elastic limit, would tend to harden the metal; but the addition of nickel (and possibly other elements) would tend to reduce the brittleness and render it workable.

The problem of finding a high, cheap alloy of steel, suitable in every particular for bridges, is now before the metallurgists and the

builders of large metallic structures; and the values of all the results probably attainable are clearly indicated in this paper; hence the onus is on the Engineering Profession to see that the necessary experiments are arranged for and thoroughly carried out, in order that the world may have at its command a new metal that will permit of the spanning of waterways which are so wide and so deep, or are so restricted by navigation requirements, as at present to defy the art of the bridge engineer.

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PAPERS AND DISCUSSIONS

DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By Messrs. H. P. Gillette, C. B. Burdick, Wynkoop Kiersted, C. W. Hudson, Allen Hazen, and F. W. Green.

H. P. Gillette, M. Am. Soc. C. E. (by letter).—With very much of the Committee's report the writer is in accord. The most important of its recommendations is that entitled the "Equal-Annual-Payment Method", which involves reduction in the property value, according to the sinking-fund formula for depreciation, and the payment of interest on the residual depreciated value. This recommendation the writer cannot support. He concedes that it is possible to apply the method, under the ideal conditions tacitly assumed by the Committee, but he regards it as impracticable under conditions which actually exist, aside from the likelihood of its being unfair. Moreover, in view of the fact that when interest on sinking fund and on depreciated value are the same, the proposed method gives identically the same rates as would result from the application of a much simpler method, the writer fails to see wherein the complex method suggested by the Committee has any advantage. Its disadvantages are certainly pronounced and many. Mr. Gillette.

The table on page 34 is used to illustrate the application of the Committee's proposed method of "Equal-Annual Payments". Column 2 gives the depreciated value, Column 3 the "depreciation during year", and Column 4 the annual interest on the depreciated value. The sum of Columns 3 and 4 is the total depreciation and interest

* Continued from February, 1914, *Proceedings*.

Mr. Gillette. for the given year, as shown in Column 6, which is seen to be a constant. Thus, at the age of 10 years, we have:

Depreciation (on $61.96)	$4.93
Interest, 5% on $61.96	3.09
Total	$8.02

This depreciation of $4.93 is the annuity that would amortize the $61.96 in the remaining 10 years of life. This calculation is entirely in accord with a principle laid down by the writer, 4 years ago, in his "Handbook of Cost Data", namely:

"The owner of a second-hand machine is entitled to such a price for it as will enable the purchaser to go on with its use and produce each unit of product at as low a cost as the average unit cost of production would be during the entire life of the machine."

Column 6 of the table on page 34 shows that this condition is fulfilled, and interest and depreciation are a constant amount ($8.02) throughout the life of the plant unit. It follows, therefore, that rates or prices for service would be unchanged, due to depreciation. For this principle of unchanged rates the writer has long contended; but, unfortunately, when a depreciated value has been used as a basis for rate-making, the rate-making body has not recognized the fact that the shorter remaining life of plant units logically necessitates a correspondingly higher depreciation annuity. In other words, rate-making bodies have invariably taken a depreciation annuity based on the full life of plant units, and then, when they have used depreciated values for rate purposes, have applied that annuity to plant units, part of the life of which has expired—a logically indefensible procedure. If depreciated value is to be used as a basis for rate-making, it becomes necessary for engineers to apply correspondingly higher rates of annual depreciation, because of the shortened time during which the owners of a utility are forced to recover their remaining capital. The public, it will be seen, will not gain in lowered rates when this is done; and will then cease to be so eager to assign a depreciated value to utility plants.

If the writer interprets the Committee's attitude correctly, he cannot see that it offers a better method than has hitherto existed for rate-making. The old plan has been to keep the capital intact, always paying interest plus profit (= fair return) on the full investment, and always providing for repairs and renewals out of earnings. In the case given as an illustration, on page 34, this end would be attained by adopting an unchanging depreciation annuity of $3.02 and an unchanging interest of $5.00 on the full investment of $100. Let us consider some of the practical difficulties that would arise were this method abandoned in favor of the Committee's method.

First, according to the Committee's recommendation, there would be no depreciation fund or replacement reserve; but the utility company would be at liberty to distribute the depreciation annuity among its owners. This carries with it the necessity of calling in stocks or bonds to the amount of the depreciation annuity thus distributed. Conversely, as soon as any renewals of plant units would be made, new securities would be issued. Such a constant flux of securities seems to the writer impracticable for many reasons, which need not be mentioned, as they will be evident on reflection. Mr. Gillette.

Second, the accrued depreciation is entirely a function of the estimated life of each group of plant units, according to the Committee's method. This assumption, however, is not justifiable except for natural depreciation. In the case of accrued functional depreciation (due to obsolescence, inadequacy, etc.), it is impracticable to estimate accrued depreciation by any formula that recognizes only the age of plant units. There is only one accurate method of estimating accrued functional depreciation, namely, by comparison of the unit costs of the product or service rendered, using, as a basis for comparison, the most economic plant unit available. This is a special case of the Unit Cost Formula for Depreciation, which the writer has recently discussed at length elsewhere.*

Even engineers are not always aware that most of the "lives" of plant units given in tables are not natural but functional lives. It is probable that the average functional life of all plant units of utility plants in America has been less than 20 years, whereas the natural lives of many of those same units is so great that as yet no engineer can fix their limit.

Thus, locomotives have had a functional life of about 20 to 25 years in America. In Europe, the functional life has been far greater, due to the slower growth of traffic and to other reasons. What is the natural life of a locomotive? No man knows. Stevenson's second locomotive is still in use in an English colliery, after more than 90 years of continual service.† Engineers have deceived themselves, and have unwittingly deceived others, by publishing tables of the lives of plant units without stating whether it was natural or social forces that limited those lives. The writer has come to have less and less confidence in statements as to lives of plant units when nothing is said as to the conditions and causes that have operated to limit such lives. Yet how rarely is a word given in explanation of those conditions and causes. To say that cast-iron pipe averages 50 years of natural life, for example, is to say what no man can prove; first, because pipe-life data are too meager to be entitled to use in striking an average; second, because short functional life has served so generally

* *Engineering and Contracting*, Oct. 30th, 1912; and *Electrical World*, Nov. 2d, 1912.

† *Engineering and Contracting*, Oct. 11th, 1911.

Mr. Gillette.

to obscure the fact that the natural life is long. Finally, the writer refuses to be bound by averages of any kind as applied to a special case unless it can be shown that the special case itself conforms approximately to the average.

The bearing of all this on the question before us is plain. The Committee suggests a method that involves the calling in of stocks or bonds, and re-issues of the same, depending on some engineers' guesses as to the probable life of many plant units, and, in most cases, the guess relates to functional life.

It will be answered that a guess as to life of plant units must be made in any event, if a depreciation fund is to be established, and that such a guess must involve guessing at functional life. This is true; but there is a marked difference between establishing a depreciation fund that shall be ample, and establishing a depreciated value that shall be fair to an investor whose capital is at stake. In the case of a depreciation fund, an error one way or the other can be rectified. If experience demonstrates, for example, that too large a fund has been accumulated, then depreciation annuities can be reduced. This can happen without injustice to the public or to the public utility company; but, let a mistake be made in calculating depreciated value, let interest distributions be based on an erroneous depreciated value, and a serious injustice may be wrought.

In the writer's opinion, a depreciation fund should be built up especially to provide for functional depreciation; also, to provide for renewals of plant units having a long natural life. For plant units having a short natural life, and where such units are of all ages, such a fund is seldom needed. A depreciation fund is, in essence, a device whereby the original investment is to be preserved intact. As a member of the Pennsylvania Public Service Commission aptly put it: "This is only a method of underwriting the investment."

As previously indicated, the third serious objection to the method proposed by the Committee arises from the necessity of great accuracy in estimating accrued depreciation. This accuracy is not limited to the time of the initial appraisal, but must be perpetual, for the repeated calling in and re-issuing of securities to avoid over-capitalization involves very accurate determination of accrued depreciation when the latter is to be the basis for capitalization. Accuracy in applying a sinking-fund formula for accrued depreciation involves opening a separate depreciation account with every group of similar plant units at the time the group is put into service. It is not correct to use "composite lives" and "composite ages" in calculating depreciation annuities for a sinking fund. Thus, a group of 1 000 poles, 10 years old, cannot be averaged with a similar group 2 years old, and a depreciation annuity calculated for the total life minus the average age of 6 years.

The fourth objection to the Committee's method is touched on by the Committee itself. A serious difficulty with its "Equal-Annual-Payment Method" arises when the rate of interest used in calculating the depreciation annuity differs from the rate of "interest" (or fair return) on the depreciated value of the plant. Then, as shown in Column 7 of the table on page 34, the "equal-annual payments" cease to be equal, and become progressively less until the plant unit is renewed, when they jump up again. The Committee thinks, however, that current repairs will rise rapidly enough to offset the diminutions shown in Column 7 of the table, but in this it errs. Suppose the plant units having the 20-year life assumed in the table are cedar poles, or any other sort of units not made up of short-lived parts. Then, assuredly, the Committee's assumption of rising repairs fails of fulfillment. Again, let the 20-year life be a functional life (not a natural life), say, of an electric generator or of a railway bridge or station, then long before the age has become great enough to cause much of a rise in the curve of repairs, the entire unit is retired. Under one of these two classes falls most of the investment in plant units of railways and other public utilities. Hence the method advocated by the Committee fails to accomplish the result that it was its aim to accomplish, namely, equal annual payments, and, therefore, stability of rates charged for service. It would seem, then, that the Committee's method results in the very predicament that other engineers have objected to, namely, variable rates for service if depreciated value is the basis for rate-making. The Committee's criticisms, therefore (see pages 48 and 49 of its report) of the views of other engineers on this matter are not entirely answered when it presents a method which is itself subject to the same criticism. Mr. Gillette.

Apparently, the Committee's main objection to using the new value of a plant as one of the elements in rate-making, arises from the belief that by so doing the public will be made to pay twice for the accrued depreciation. The writer's experience thus far has been that the public has rarely or ever recouped the railways and other utility companies for the accrued depreciation. Depreciation funds seldom exist, and, where they do, they still more seldom equal the accrued depreciation. What is more significant, however, is the almost universal development cost, or accumulated deficit in fair return, that railways and other utilities disclose when their accounts are properly analyzed. When, in calculating development cost (= the "going value" of the Wisconsin Railroad Commission), a depreciation annuity is allowed as a part of the past operating expenses, sufficient to provide for the accrued depreciation shown by an appraisal, it is a rare occurrence when the development cost does not more than equal the accrued depreciation. This is one of the surprises that a much deceived public is to receive. Muck-raking articles based on the flimsiest of evidence,

Mr. Gillette. grossly distorted, have created a very general impression that nearly all public utility stock is badly watered, and that enormous profits have been universal. Engineers, who should know better, are not entirely free from a bias created by this false "testimony".

Accrued depreciation receives its proper consideration in calculating development cost, or the cost of establishing the business. The writer has discussed this matter at some length elsewhere* and, therefore, will not go further now than to caution against a blind adoption *in toto* of the Agency Theory on which the development-cost calculation rests. Although it is wise to determine what would be the total investment (physical and non-physical) according to the Agency Theory, it should not be forgotten that the theory has evolved slowly and is comparatively new. To what degree a new theory can rightly be made retroactive is a matter for serious consideration.

These and other difficulties confront rate-regulating bodies which aim to be fair. As yet there seems to be no escape from the necessity of weighing all the facts bearing on value as well as on cost, as outlined in Justice Harlan's celebrated decision (1898) in the Smythe *v.* Ames case.

The Committee's report indicates an effort to compromise between the Agency Theory and the Replacement Theory, by taking past conditions and present prices. Is it not better not to mix the theories, but to present the results of each theory complete and logically derived? Then the final "rate-making value", which is likely to be a compromise, will be arrived at after weighing all the evidence. The engineering mind usually rebels at a compromise of any sort, but in appraisal cases compromises are almost inevitable, primarily, because we are in a transitional period. The old Competitive Theory is rapidly being modified by the new Agency Theory of rate regulation. Not only does each theory have its merits, but it is impracticable to make the new theory entirely retroactive without working grave injustice in many instances.

Mr. Burdick. C. B. Burdick, M. Am. Soc. C. E. (by letter).—This report is particularly opportune, for we have just entered an epoch of rate regulation; and the valuation of the railroads lately undertaken by the Interstate Commerce Commission has directed many able minds to the subject for the first time. It is not surprising that, under these circumstances, many ideas strongly advocated are very discordant with one another. To the casual observer it might seem that the principles of valuation are in a very chaotic state, or, as the Committee expresses it, that they are in a developmental stage. The principles involved, however, are much more clearly established at this time than appears on the surface.

* *Engineering and Contracting*, June 26th, 1912.

Mr. Burdick.

The Committee has apparently attempted to formulate a procedure for fixing an equitable value that may be used in the matter of rates, by considering each item of value by itself, as to its equity or inequity, on the assumption that, if the parts are equitable, the sum of the parts must be. The defect in this procedure lies in the impossibility of determining what is equitable in most cases until after all the parts have been summed up and considered in the light of all facts having a bearing on value. When the sum is obtained, it represents nothing in particular. It is neither present cost of duplication, nor investment, and probably very imperfectly represents value.

It will greatly clarify the discussion to decide at the outset whether the fair return to be established shall apply on investment or on the value of the property.

If public utilities were not yet established, it would seem entirely equitable to all concerned to provide that the rates should be based on a fair return on reasonably wise investment. The public could have no cause for complaint, and the investor, being reasonably sure that his investment would be protected, would probably be content with a low rate of return. Eras of high and low prices would affect injuriously neither the investor nor the public. Those looking for large profits would invest elsewhere. However, no such situation exists. In practically all cases at this time we are dealing with established properties, nearly all of which antedate the epoch of rate regulation and the Court decisions based on public policy that have made rate regulation possible. The Constitution of the United States provides that private property shall not be taken for public use without just compensation.

The Courts, which must be our guide in the interpretation of the law, are very firm that rate regulation shall be based on the value of the property concerned and, therefore, in valuations for rate-making purposes, it is the duty of the appraiser to determine value as nearly as can be done under the circumstances existing at the time of the inquiry.

How, then, shall we measure this value? The Courts are authority for the belief that cost of duplication under present circumstances is an important measure; that the actual investment should be considered; that even the stocks and bonds and the revenues derived from operation are not without the pale of consideration; that all these should be given their proper weight, under all circumstances, in throwing light on the value. Further, that the value must be reasonable to the buyer and seller, and, in the case of rates, that they must be equitable under all circumstances to the owner and the public.

In various cases involving property the Courts have held that the owner is entitled to the accretions in its value from whatever causes they have arisen, and if the property has depreciated that must be taken into consideration, if for any reason the property must be

Mr. Burdick. valued. The owner must receive the benefit and suffer the loss of value. In taking his property for the public use he must be remunerated for the property as it exists at the time taken. In fixing values for the purpose of rates and values for the transfer of ownership, it would seem that value in the two cases is one and the same thing, the fair present value of the property; and, so far as observed, the Courts have made no important distinction in the values for these two purposes.

For the same reasons of public policy that permit rate regulation, it is to the public interest that wise investment should be reasonably protected. The property must be allowed to earn rates that will enable it to be self-supporting. It is in the interest of public policy. Unless the property may earn such rates, the service cannot exist. Therefore the Courts have expressed an interest in the amount that has been invested and are loth to fix values, or rates of return, or both, which would tend to discourage investment in like enterprises. For the same reason inquiry as to the stocks and bonds is pertinent, and though the individual security holders cannot be protected against the operations of frenzied finance, the reasonableness of the financial operations can be determined in the light of the figures on investment and cost of reproduction. What light would be thrown on the matter by an estimate of the cost of reproducing the property at present prices under the "conditions existing at the time the various portions of the property were built?" It would throw light neither on the investment nor on the present reproduction cost of the property.

It would seem that the Committee has shown a leaning toward investment as a measure of value for purposes of rate-making, but through the influence of Court decisions, which are numerous and strong, had been forced to the adoption of present prices in the estimates of value. This, it is believed, has placed it in the indefensible position (Conclusion 7) of reproducing the property under the "conditions existing at the time the various portions of the property were built, but on the prices prevailing at or near the time of the valuation." This amounts to assuming part of the circumstances in the past and part in the present, for unit cost is as much a circumstance of the time as any other element subject to change with time. In justification of this conclusion, a case was cited of a reservoir in the construction of which a railroad track was necessarily moved to a new location, the cost of removing the track being properly included in the value of the reservoir. The conclusion is further applied to exclude pavements laid over water pipe after the pipe is laid, and to include the extra cost of "piecemeal construction" in computing the value of water mains. It does not require this general conclusion to justify the inclusion of the railroad track. If we reproduce under the circumstances of the present, we are bound to assume reasonable conditions,

Mr Burdick.

and if a railroad formerly occupied the site of the reservoir, it would be entirely reasonable to assume that it would necessarily be removed in reproducing at the present time, unless there should be good reason to assume otherwise. In reproducing such a reservoir, it would be necessary to assume the contour of the ground, and one could be guided in such assumption only by approximating conditions known to have existed in the past. In this case the past is a guide to reasonable assumptions for duplicating under present circumstances. The cases of pavements and "piecemeal construction", the exclusion of the former and the inclusion of the latter, in the sum total of value, are reversions to past cost or investment, and would seem to have no place in present cost of duplication or present value.

It would seem most logical in the valuation of these properties to pursue one logical method through to the end. If it be the cost of duplication at present, then reproduce under present circumstances, and when the figures are summed up, one will have a total that represents something tangible—the cost of duplication. If it be past cost, an estimate of the investment, then follow past cost consistently to the end, and the total will represent investment. Neither total will necessarily represent value, but the two will point strongly toward it, particularly if they are not far apart.

The Courts state that the rates produced must be equitable both to the owner and to the public, and it is here that an opportunity is offered to test the reasonableness of the totals previously determined. If fair rates of return applied to the total cost of reproducing the entire property necessitate rates that are reasonable to the public, then all the equities are satisfied. If the rates necessitated are unreasonable, then the value or rate of return must be reduced to such amount that the inequity is distributed in a just way between the owner and the public.

In some cases the foregoing procedure is difficult, but in most cases an equitable decision is the result of industry and intelligence in marshaling all the obtainable facts and common sense in their application to the problem particularly in hand. It is not a task for a novice. It requires broad experience in every important phase of the property to be valued, an open mind, and some considerable acquaintance with the subject of valuation in general. A realization of the truth stated by the Committee, that rates too low are unjust to the public, is helpful in establishing justice.

The Committee frankly states, after pointing out the necessity of following the Courts:

"And while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

Mr. Burdick.

Is this a wise position? Has the Committee had the opportunity to consider the matter sufficiently to warrant it in recommending procedure at variance with the law as established by the higher Courts? The law is the outgrowth of the experience of centuries. It is necessarily modified from time to time as new conditions arise. A great concession in favor of the public was made when rate regulation was permitted in the face of existing contracts, but the Courts stand firm as a rock against some of the views advocated by the Committee.

Mr. Kiersted.

WYNKOOP KIERSTED, M. AM. SOC. C. E. (by letter).—The writer has been very much interested in studying this report, which is certainly instructive; but, before the Committee makes its final report, it is hoped that the application to a composite property of the Equal-Annual-Payment Method as well as of the other recognized methods of computing depreciation will have been shown.

The question of rate-making requires the ascertainment, among other things, of a fair earning value of a public utility and a rate of return thereon which is fair to the investor and to the public, and which will provide a fund for replacements and renewals to an extent which, when invested in replacements, will maintain the property in an efficient condition and will protect the original investment. The decision of the United States Supreme Court in the Knoxville case appears to confine the fund for depreciation to replacement of units to an extent which will maintain the original investment unimpaired. Such replacements are expected to be made periodically as needed to keep the property in an efficient operating condition, and when so made it is difficult to escape the conviction that a property thus maintained should be entitled to a return on substantially a 100% value. The writer expressed this view a little more in detail in his discussion of the paper* by John W. Alvord, M. Am. Soc. C. E., on "The Depreciation of Public Utility Properties as Affecting Their Valuation and Fair Return". To what was said in that discussion may be added the suggestion that the earning value of a composite property, made up of the earning value of its units, is not affected by consideration of age as long as repairs of the unit and replacements of parts of the composite property receive timely attention. A unit, retained as a necessary part of an efficient property, possesses a 100% earning value, whether old or new, quite as much as the composite property itself. The question of accrued depreciation, as applied to either the value of the unit or the value of the composite property, does not arise for consideration in a rate case as it does in a valuation for purchase and sale.

The life of a unit of a composite property depends on such a variety of conditions as to be indeterminate with precision. The best evidence in this regard is that furnished by a study of the cost of

* *Proceedings*, Am. Soc. C. E., February, 1914.

operation, maintenance, and replacement of properties which have been in use long enough to afford a wide range of operating conditions. Such information collected from each class of property, properly analyzed, should be the best guide as to the proper rate of return on an investment to cover the cost of replacements. Doubtless a large amount of information of this character has been accumulated through various sources, which should aid materially in the solution of such problems. Mr. Kiersted.

Although the rate of return as interest or dividends, and as profits, may be practically uniform for some classes of property in particular districts, it does not follow that the rate of return to supply a fund to take care of replacements shall be correspondingly uniform. For instance, a public utility in a rapidly growing city requires proportionately heavier expenditures in replacements during the earlier than during the later periods of its existence, whereas the reverse may be true of a slowly developing utility in a slowly growing city. The rate of return for replacements in one case should evidently be more liberal than in the other. Moreover, many physical conditions and service requirements will have a modifying influence to an extent which requires a close analysis of the individual property.

Any method dealing with a rate of return on a public utility, considered as a whole, must be based finally on equal annual payments, and will be divided essentially into two parts, namely, a rate of return to the investor as interest or dividends, and as profits, and a rate of return to cover replacements. As long as the rate of return is equitable for both purposes, it does not matter what the detailed analytical methods may have been in reaching a conclusion, if that conclusion is substantially correct. In order to reduce the probabilities of error of individual judgment as much as possible, it were perhaps as well that the analytical methods, taking into consideration the units of a composite property, should embrace the use of the several rules or mathematical formulas rather than that of only one rule or formula.

The valuation of portions of a physical property on the basis of present value under original conditions, and other portions on the basis of present value under present conditions, introduces an uncertainty as to whether such a distinction should be made, and, if made, whether the reasons therefor are sufficient to justify the distinction. Furthermore, the unearned increment included in an item of property, particularly real estate valued under present conditions, raises the question as to the extent which this unearned increment of value is to be considered in a rate case, and whether, if considered, it should earn a rate of return equal to that of other portions of the physical property, barring a return for replacements. Still further consideration is open to the question of whether such an item of property as real estate, possessing value independent of the public utility with which it may be associated, by reason of urban surroundings and asso-

Mr. Kiersted. ciations, is subject to the same degree of hazard of ownership as that attaching to other items of property possessing practically no value except as a part of the public utility. If the hazard in the one case is less than in the other, should the owner be entitled to as great a return on the portion of his investment involving little or no hazard as on that portion involving a positive hazard?

Mr. Hudson. C. W. Hudson, M. Am. Soc. C. E. (by letter).—It appears to the writer that the Committee should have confined its report to the field for which it was appointed. Undoubtedly it will be conceded that engineers have special knowledge on the subject of cost of reproduction of the physical elements of public utility property. Assuming that the Committee has been able to formulate principles and methods which will be of use in determining the value of the property of railway and public utility corporations, it by no means follows that it has been or that it will be able to show how the values thus determined may be used to fix rates.

In the case of a public utility engaged in supplying one commodity to a limited district, the cost of the service may bear quite a close relation to the value of the plant, and in so far as that value affects the reasonableness of rates for service, it might be taken as the cost of reproducing new, the cost of reproducing new less depreciation, or the amount of capital invested. For the case of railway property, which is used under such varying conditions and, in the aggregate, represents nearly one-sixth of the wealth of the nation, it will not be possible to establish to the satisfaction of a majority of the members of this Society a working relation between value and rates.

The making of the tariffs of the railroads is in itself such an intricate and extensive subject, involving as it does the stability of our entire economic structure, that the writer does not believe the Committee was warranted in trying to formulate principles of valuation for rate-making. It is certain that a careful study of the report does not show how it would be of the slightest service in the preparation of proper railway tariffs.

Numerous reasons may be cited to show that even the cost of the service, which includes the value of the plant, should not always determine the reasonableness of a rate. An important requisite is that the traffic must move; that is, that the rate for the service shall be low enough to allow it to move.

The Committee does not appear to include the most valuable elements in its valuation for rate-making. To quote from the report:

"Justice and equity require that in any regulation of rates fixing the amount which may be earned by a public service corporation there shall be taken into consideration two distinct features:

"First, the annual return covering interest and profit, to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment. Mr. Hudson.

"Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

In other words, the Committee would be willing to insure the railways an attractive earning capacity based on these two considerations. To the writer's mind, the owners of railway property could lie down and rest in perfect repose as far as any fear as to what their competitors were doing was concerned. Every business enterprise in order to succeed must have brains, honest labor, and loyalty back of it. To base a rate for service on any consideration of value of the physical plant alone, leaving out the more important factors, would be a great economic mistake, and it will be a serious error for the United States to do any such thing.

It also appears advisable to the writer to separate the rules and methods for valuing public utility properties from those for valuing railway properties.

He does not believe it will be possible for the members of this Society to come to anything like a general agreement as to how railway rates are to be fixed. Very few of them have an extensive knowledge of the subject, nevertheless, there are probably many members who are willing to take it out of the hands of experts. The writer wishes to express his conviction that the Society should not publish any rules for valuation for the purpose of rate-making.

Allen Hazen, M. Am. Soc. C. E. (by letter).—The writer has the feeling that under the heading, "Present or Original Conditions", the Committee has given more attention to the history of the plant than is fairly justifiable. For instance, in estimating the cost of reproduction of a water-works plant, the cost of cutting through the paving of the street over it, and replacing such pavement after laying the pipe, is unquestionably a part of the cost of reproduction, and must be included in any true estimate of it. The Committee, however, would exclude all that part of the cost which relates to paving laid after the pipe was placed. The writer is unable to see that the question of whether the pipe or the paving was laid first has anything to do with the cost of the reproduction of the property as it now stands, or with its value. It would cost just as much to replace the pipe in one case as in the other. The value of the property, as measured by its capacity for useful service, is identical in both cases. Mr. Hazen.

This procedure, together with others in the report, seems to have been adopted by the Committee in an endeavor to get at something

Mr. Hazen. that may be taken as the equivalent of "invested capital" to be used in place of the actual amount of the investment, which may be unknown. The impression is given that the Committee had in mind something like the following: What amount of capital might reasonably have been invested in the enterprise, assuming that the prices of the last few years had obtained during its entire history? This idea seems to have displaced to a considerable extent that of the cost of reproduction.

Going a little further, the distinction between cost and value does not seem to be made, and the whole report seems to be directed to consideration of cost. So far as the writer can find, value is not considered in it at all. It may be that in a subsequent report the relations of cost to value will be discussed. Otherwise, it would seem better that the title of the report should be changed, and that cost, or cost of reproduction, should replace the word "valuation" in it.

Mr. Green. F. W. Green, Assoc. Am. Soc. C. E. (by letter).—This report is a notable addition to the literature on one of the momentous matters of the day, and the comprehensive, and withal temperate, treatment by the Committee of so difficult a subject is, in every way, praiseworthy. A thorough discussion undoubtedly will assure a more perfect understanding of the matter; and a clear understanding must precede a perfect judgment.

The attitude of a very considerable number of persons toward the railways as a whole is suspicious, if not hostile; it seems to have been reached by a most unusual method of logic: (1) A, B, ... Z are railway companies; (2) A and J have been corrupt, inefficient, and arrogant; Conclusion: A, B, ... Z have been corrupt, etc. This is a general conclusion from a specific premise.

Again, assume two roads, X and Y. Road X was honorably conceived, constructed, and operated. It replaced its equipment and maintained its plant from adequate reserves from earnings, and it was not necessary for it to capitalize such replacements and maintenance. Road Y pursued the opposite policy. Should X be condemned for the sins of Y?

Two issues present themselves in connection with the consideration of the unearned increment in land values. If other land owners are permitted to enjoy this increment, may railways justly be deprived of it? If railways are not to have equal benefit of the laws, why does the State assess taxes on the basis of the increased value? This brings up the matter of the railway charter. In the usual form, it is in the nature of a contract between the State and the corporation. On the part of the State there is the obligation to do certain things, namely, the grant of authority to buy and sell, to contract, to exercise the right of eminent domain, to locate, construct, and operate a railway, to sue and be sued, etc.; and, on the part of the corporation, in return for

these grants, it agrees to do certain things, namely, to furnish adequate transportation service, at reasonable rates, and without discrimination; and this implies the development of new country, the increase of private and public wealth flowing therefrom, and the foundation for a large unearned increment for those land owners who may be fortunate enough to be located near the new development. Some writers aver that the increment in railway right-of-way values is the result of the community development; but if this is so, what caused the community development in the first place? Mr. Green.

As to the various Court decisions, it is the writer's belief that in some instances these have erred, either from a failure to present the issue to the Court in terms of the Court's comprehension, or from failure of the evidence to clarify and illuminate the issue with sufficient detail. It is no uncommon experience for engineers to have great difficulty in communicating a more or less technical matter to eminent lawyers in such a way that it will be clearly understood. Again, there appear to have been some cases, at least, in which the issue was confined to the consideration of the strictly legal aspects of the case, rather than to the matter in its economic aspects.

The matter of efficiency of operation and special advantages of location is of absorbing interest. If the well-located line is denied a reward for efficient management, there will be no incentive for it, and the public will be required to contribute to the extravagance resulting. Again, if the rates are to be measured by the amount required to sustain the strongest line, what will become of the weakest? If the rates are adapted to the necessities of the weak line, will not the strong line enjoy inordinate profits; or, if a fat rate is permitted on the weak line, and a lean rate on the strong line, will not development move along the line of least resistance, and population and traffic increase on the strong line and decrease on the weak line? If so, will there not be a discrimination established between communities, which will produce much more public discontent than under the present alleged unsatisfactory conditions? It is along these lines that the real difficulties of applying the theory of making the rate a function of the valuation become apparent. The Committee has very wisely remarked that other considerations should enter into the subject of rate-making. It may be pertinent to mention some of them: The competition of carriers; the competition of commodities; competition of communities; value of the service to the patron; cost of performing the service; density of traffic; direction of traffic; volume of traffic; carload minima and maxima; slow or fast movement required; and insurance as carriers.

As to depreciation: Some hold that rates should be based on the depreciated value of the property, unless and except, the depreciation fund in cash is kept on hand intact, and so denominated. The usual

Mr. Green. argument is that, if such fund is not thus kept and labeled, there is a return of capital to the owner of the utility, and hence the capital invested in the first instance is withdrawn and available for reinvestment. It is to the user's interest that the property be maintained, both with regard to adequacy and economy. As long as efficient service can be given, the user should expect to pay to the owner as much for the same service on an old road as on a new one; and as there is no requirement that maintenance be paid for out of any other than the general funds of the owner, and further, because adequate maintenance is, or should be, a prior lien on such funds, there would seem to be no justice in making the rate fluctuate with the variations of physical condition. Moreover, if it is conceded that there is in effect a return of capital to the owner, does not this carry with it the implication that it would be permissible for the owner to capitalize maintenance charges when made, and would this not be in violation of the principles of accounting established by the Interstate Commerce Commission?

It is to be hoped that in due time will come recognition of the economic truth that the welfare of all depends on the just and equitable treatment of each unit in the body politic.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion.*

By Messrs. Nelson P. Lewis, John C. Trautwine, Jr., William Goldsmith, George A. Ricker, George W. Tillson, Robert A. Meeker, R. E. Beaty, William M. Kinney, D. B. Goodsell, H. W. Durham, Herbert Spencer, T. Hugh Boorman, and Philip P. Sharples.

ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

Nelson P. Lewis, M. Am. Soc. C. E. (by letter).—It being admitted that an engineering organization is required to deal with highway work, it follows that the details of such an organization should be determined by engineers. The time has passed when engineers should be expected to carry out municipal work of any kind when they are told, as they are told, and because they are told. Municipal engineers must assume the responsibility of determining, not only the kind of organization through which this control is exercised, but the general policies of a city with respect to such work. Every city needs a definite plan and programme. The need of a comprehensive plan for a street system is now generally recognized, but it is not as generally recognized that this should be the work of the engineer. A paving programme is just as essential. The policy of most American cities has been to leave the kind of pavement to be laid on any street to be determined by the abutting property owners, on the ground that, as they are to pay the bills, they are entitled to say what kind of pavement they should have. Such a plan cannot result in a well-paved city. By a well-paved city is meant a consistently paved city, that is, a system of pavements in which the surface on each street is determined by the grades, the traffic, the development of the abutting property, and its ability to pay. If grades permit, it is desirable to have Mr. Lewis.

* Continued from February, 1914, *Proceedings*.

Mr. Lewis. the same kind of pavement on a continuous thoroughfare, and it is very desirable that the driver of a team should be able to follow a pavement on which horses can travel under any weather conditions from any part of a city to any other part.

There should also be a definite plan for the location of sub-surface structures. The building of pipe subways has frequently been urged as the only means by which an orderly arrangement of underground structures can be secured and the constant opening of the street surface prevented. When a new street is being built or an old street is being widened and entirely reconstructed, the placing of pipes and conduits in subways may be practicable, but, unless all these pipes are new, and unless the greatest care is exercised to prevent leakage, especially of gas, the risk of carrying out such a plan is considerable. Paris no longer permits electric wires to be placed in the sewers, where the water and gas mains are accommodated, but separate electrical conduits are usually placed under the sidewalk. This, again, is a subject for careful engineering investigation, and is worthy of assignment to a place in an engineering organization dealing with highway work.

In the opening discussion, the city highway bureau was aptly described as the principal show-case of the city government, the pavements representing the goods in the window. It follows that there is no department of municipal work where inefficiency is so sure to result in criticism as in the highway bureau, but the organization to deal with this branch of municipal work should not be limited to the mere laying and maintenance of pavements and their cleansing; it should extend to the planning of the lines and grades of the streets, the selection of the type of pavement for each street, a rational method of financing the cost of a street improvement, and also the proper placing and care of the underground structures, which do not appear in the show-window, but which, nevertheless, are an important adjunct to every street.

Mr. Trautwine. JOHN C. TRAUTWINE, JR., ASSOC. AM. SOC. C. E.—Mr. Durham's discussion calls attention to what may be called two systems of stratification, in the matter of highway organization: (1) The vertical system, in which each of the several geographical divisions of a large city constitutes, in effect, a little sovereign state, with its own highway organization, independent of those of the other divisions; and (2) the horizontal system, in which a single highway organization presides over the highways of the entire city, including all its sub-divisions; as Pennsylvania's blanket of "new red shale" overlies the upturned edges of many much older formations.

Now, although Mr. Durham expressed his preference for the second, or horizontal system, the vertical system appears to obtain in

New York City, where the Borough of Manhattan, the Borough of Queens, the Borough of the Bronx, and the Borough of "What-not," each constitutes apparently a little highway principality, with sovereign powers over its own affairs, and permitted to act (if it chooses) in imperial disregard of the arrangements and interests of the others; so that, conceivably, two adjoining boroughs, in an outburst of local patriotism, might so revise the alignments and grades of their highways that those of one borough should have no physical connection with those of the other. Mr. Trautwine.

Now, to paraphrase Sterne (who has been quoted in this discussion) the speaker submits that "we do these things better in Philadelphia." Mr. Connell will correct the speaker if he is wrong; but he is under the impression that Mr. Connell reigns supreme over the highway organization of that city, and that there are no highway chiefs of Manayunk, or of Chestnut Hill, or of Frankford, or of Southwark, empowered to regulate highway matters, in their several districts, without consulting him.

Philadelphia has the horizontal system of stratification; and this is in accord with the modern tendency to centralization, with its necessarily higher efficiency.

The programme for these meetings mentions State and Municipal Highway Organizations. The speaker ventures the prediction that, within 50 years, we shall have a National highway organization, administering the streets and highways of the entire nation, and of its cities, from whatever is then the national capital; and that, within a century, we shall have a World highway organization, similarly administering, from a central office, the streets and highways of civilization —with universal benefit on a scale now undreamed of.

The same will be true, of course, of matters like water supply and sewerage, now in general conducted (on the vertical system of stratification) by each city for itself; although Massachusetts (in the advance, as usual) has set a notable example in the formation of its Metropolitan Water and Sewerage Board, which regulates these matters for Boston and its neighboring communities, each of which formerly looked out for its own interests, mostly without reference to those of all the others.

Of course, it is not to be expected that the suggestion of a national and a world highway organization will be looked on with favor by incumbents of municipal offices which, under such organization, of course, would have to accept subordination in exchange for their present sovereignty, unless indeed from those who felt pretty sure of being called to the national or world chiefship.

William Goldsmith, Assoc. M. Am. Soc. C. E.—There is one detail in Mr. Connell's discussion which has proved of vital importance to the organization of the Highway Bureau in the Borough of Man- Mr. Goldsmith.

Mr. Goldsmith. hattan. This relates to patrol inspection. In past years civic organizations and private citizens have complained repeatedly about the poor condition of the pavements in Manhattan Borough, and particularly with reference to the length of time elapsing between the opening of a pavement and its final restoration. These complaints culminated in an investigation by the Commissioners of Accounts during 1906 and 1907. Their report showed:

First.—That there was no system which made it possible to follow up cuts made in pavements so that they could be quickly restored.

Second.—That the responsibility was not fixed, and the shifting of blame from one to another was not only possible, but was the case whenever difficulties arose.

In 1910, under Borough President McAneny, steps were immediately taken to remedy these conditions. The Bureau of Municipal Research, working in conjunction with the Bureau of Highways, inaugurated an inspection system, which, during the last 2 years, has worked admirably. Although this is a detail in highway organization, it is vital to success, and is worth while noting.

The city is divided into districts, for each of which there is a patrol inspector. His territory covers about 10 miles of pavement, there being about 450 miles of paved streets in Manhattan. It is the inspector's duty to report at least once a week regarding the general condition of the pavements, and to report promptly on all street openings, building operations, and encroachments for which permits are issued.

In order to give a clear idea of the inspection system referred to, take, for example, a plumber's street opening. The plumber must be licensed (pass an examination as to fitness, before the Board of Plumbing Examiners), and must execute a $1 000 bond with the city for the proper performance of his work. He comes to the Permit Division, which is under the direction of the Bureau of Highways. A technical man, called the Supervisor of Permits, is in charge of this Division, and keeps in direct touch with contemplated pavement work, sewers, or other constructions; sees that sub-surface corporations do their work in streets previous to resurfacing; that all departments are notified of contemplated constructions; and approves every permit before it is issued. The plumber makes a cash deposit covering the probable cost of pavement restoration. This payment includes an inspection and overhead charge (which amounts to approximately 50 cents per sq. yd.). He must also make a cash deposit of an equal amount, which is retained for 6 months, to insure the city against any settlement of the pavement, due to improper back-fill, or other poor workmanship.

As soon as a permit is issued, triplicate inspection forms are made out on a typewriter in the Permit Division. These forms provide space to give the inspector in the field all the information necessary for a

Mr. Goldsmith.

proper inspection of the work. These forms are called: (1) a "Preliminary"; (2) a "Tickler"; and (3) a "Final".

The "preliminary" report is given to the inspector, whose duty it is to see that the conditions of the permit are complied with, to report thereon what work was done, and, by actual dimensions and sketch, show just how much pavement was disturbed, and the quantity to be restored and its exact location.

The "tickler" and "final" forms are filed in a cabinet provided for that purpose in the main office. When the "preliminary" report is received from the inspector, the "final" is sent to the pavement contractor, who honors it as an order from the city to restore the pavement. The contractor hands this "final" to his foreman, who takes it out on the work and makes the necessary repairs in amount and location as shown.

With these repair gangs there is another set of inspectors, called maintenance men, who are entirely independent of the patrol inspectors. The foreman of the repair gang hands this "final" report form to the maintenance inspector assigned to his work, and it is this inspector's duty to see that the pavement, of the exact amount and location shown on the inspection form, is repaired according to specifications. He then records the actual amount repaired and signs this "final" form in a space provided for that purpose. These maintenance inspectors are notified by a chief inspector, about 18 hours in advance, just where their work will be on the following day.

After the day's work is completed, the inspector fills out and signs the "final" reports, and mails them to the main office. In this way, an independent check is obtained, and collusion is almost impossible between the inspectors and contractors as to the area of pavement to be restored. The paving company then presents its bill for this restoration work to the city, and a clerk in the office, uses the "final" reports as a basis on which to approve the bills.

During 1913 an experiment on patrol inspection was tried, the object being to reduce the number of patrol inspectors and put them where the work was urgent. The territory north of 59th Street and west of Fifth Avenue, in Manhattan Borough, was patroled by one inspector in an automobile. Previously, this section had been patroled by twelve men. The difference in cost was:

12 Inspectors at. .$1 200 =		$14 400
as compared with:		
1 Inspector at...	1 800	
1 Chauffeur at. .	1 200	
Maintenance and depreciation of automobile	1 000	
Total		4 000
Yearly difference, or saving......................		$10 400

Mr. Goldsmith. This covered a mileage of approximately 132 miles and gave a patrol inspection cost per annum of \$30 + per mile, as compared with \$109 + per mile by the old system, showing a saving of \$79 per mile per annum, or a total of \$10 400 per annum.

Although this scheme worked out well in the district where it was used, it probably would not be as effective in heavy traffic sections, where the territory covered would be limited on account of congestion and where the speed of the automobile would not be an advantage.

The inspection and follow-up system referred to, which includes a number of patrol inspectors and an independent number of maintenance inspectors, has the following advantages:

(1) It places individual responsibility on the inspector on the work, as the reports must be signed by him;
(2) It makes the shifting of responsibility impossible;
(3) Possible collusion between inspector and contractor is out of the question;
(4) Experience shows that better results are obtained.

The fourth statement is easily verified by a comparison between the present state of the streets in the Borough of Manhattan and their condition under the old system.

In a highway organization, the details of the inspection system are vital to a successful administration, because the success or failure of highway work depends on the field inspection. The speaker believes that the system outlined is feasible, covers all necessary requirements, and, if properly supervised, will get rid of many of the difficulties with which a city highway engineer must contend.

Mr. Ricker. George A. Ricker, M. Am. Soc. C. E.—The speaker is glad of the opportunity to say a few words about the reorganization of the New York State Highway Department, and the methods that were followed in selecting the Division Engineers. When Commissioner Carlisle and the speaker undertook this reorganization there were only three regular Division Engineers on the work, and, as the new Highway Law commanded the redivision of the State into nine divisions, there were six vacancies to fill. It was somewhat troublesome to think of how to secure competent engineers for these divisions who, at the same time, would be good executives, for it must be borne in mind that these divisions are very large, each containing a mileage of roads as great as the whole of Massachusetts.

It was thought to be practicable to secure by civil service examination and appointment men who had the necessary personality, men with executive force and the ability to command. By permission of the State Civil Service Commission a plan of co-operation with the Highway Department was established in the preparation and conduct of an examination, which consisted of three parts. First, an oral

test (and there is point in its being placed first); second, two theses, one on the subject, "Essentials of Highway Construction," and the other on any large work in which the applicants had had experience; the third section consisted of a sworn statement of each man's experience. Mr. Ricker.

There were about 160 applicants. One by one the men were called into the office of the Civil Service Commission on a given day, and two members of the Board of Consulting Engineers and the speaker sat with the Commission. An attempt was made to put the candidates at ease by the attitude of the examiners, and by asking them unimportant questions until any nervousness which they felt was overcome. After a few minutes' conversation the leading questions, calculated to bring out the characteristics of the candidate, were put. It was assumed that the candidates were ready with technical matter, but technical questions were avoided, as the examiners were quite sure that the men were well equipped along that line, and if not, that would be found out in the later tests.

The object of placing the oral test first, it will readily be seen, was because men of executive force and strong personality were needed. They might have all the technical knowledge in the world, all the education and experience necessary, but, if they could not control men, they would not make good Division Engineers. By this test it was possible to exclude from the examinations that would follow men who were lacking in necessary personality to make good executives. Some fifty or sixty men were selected, whom it was thought were very desirable, and they were instructed to enter the other tests. It should have been stated, by the way, that as each man left the room the examining board agreed on his mark before the next man was called.

The entire board read and criticized the theses, and, as a result of the complete examination, the Civil Service Commission presented a list from which six men were selected from among the first seven men: one from the Board of Water Supply, three from the Barge Canal organization, one from the Highway Department, and one who had had six years' experience in the Panama Canal work. These six men, with the three from the old organization, constitute the Division Engineers of the reorganized Department. All these men are members of this Society, and all are also graduates of recognized engineering schools, the several institutions represented being the Massachusetts Institute of Technology, Cornell, Rensselaer, Polytechnic, and Marietta College (Ohio).

Following this start in the reorganization, each of the Division Engineers has been supplied with an Assistant Division Engineer, the latter from promotions in the Department. An examination was held, following a plan similar to that outlined for the Division Engineers, and twelve men from the first thirteen were appointed, some divisions getting two men on account of their size and the magnitude

Mr. Ricker. of the work. These Assistant Division Engineers, technically known as Resident Engineers, were taken, with one exception, from the top of the list down. The one man passed, as in the case of the Division Engineers' list, was not passed on account of any political reasons. Politics had absolutely nothing to do with the selection of any of these men. They were chosen and appointed before any one of them knew that they were being considered, so that nobody had any chance to "see the Commissioner" or the speaker regarding them, and they had no opportunity to send anybody on their behalf if they had wanted to, and, as far as the speaker knows, none of them ever thought of doing such a thing. Thus it is seen that the reorganization is purely on a merit basis.

Arrangements are now being made for the selection of Assistant Engineers to place in charge of counties. The Division Engineers have recently met and discussed the list of available men presented by the recent promotion examination, and have agreed among themselves as to the disposition of these assistants throughout the State by counties.

An attempt is being made to establish something approaching a military organization, and the Division Engineer in complete charge of his division will be supported and sustained. Politics will have absolutely nothing to do with the work if it can be prevented, and it is thought that it can. Commissioner Carlisle, although not an engineer, has engineering sense, and in a marked degree possesses executive and administrative ability.

In January, 1914, there was established a system of weekly reports, which are really time sheets, so that every man in the Department reports to the Bureau head and Bureau heads send to the Department head a record of what every man is doing every day. Thus it is known where the men are, and what they are doing, and in that way one can readily keep track of the force.

FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

Mr. Tillson. George W. Tillson, M. Am. Soc. C. E.—This subject is of great importance; it has been discussed for a great many years, and will be discussed for many years to come. The speaker has made the statement several times that in the preparation of plans for road and street work engineers were more behind than almost any class of constructive workers. That statement will have to be modified from time to time, as the subject is receiving so much discussion from year to year and from day to day.

It will be remembered that Mr. Green started out with a proposition that what was said in the discussion must be new. That is a

very difficult requirement, and, under it, the speaker feels that he cannot qualify, especially after listening to Mr. Green's discussion; he thinks that a man who could talk for ten minutes and say nothing but what was new to every member would be a wonder. If the speaker can suggest one idea that is new, he will be satisfied. The point he wants to make is that the ideas which govern roadway and street construction and the design of both must not be said once and then laid aside. In order to make inspectors and road supervisors entirely familiar with them, they must be reiterated, told over and over again, said in a new way, or said in the same way, until they know them as thoroughly as they know the alphabet. Mr. Tillson.

In the design of work of this kind, it seems that the first thing an engineer must do—and the whole situation will be discussed in as general a way as possible—is to make himself thoroughly familiar with the work that is before him. If an engineer is to design a bridge, if an architect is to design a building, the first thing to do is to find out what will be required of this bridge or this building. The first thing the engineer must do is to study the requirements of the road he is to build or the pavement he is to construct.

In doing that, he has a great many things to consider. That is brought out to a great extent by Mr. Green. The character of the traffic: Until the last few years no one has considered it necessary, or, if necessary, that it was worth while, to have a traffic census, either in city streets or country roads. No longer ago than December, 1912, at the Road Convention in Cincinnati, after an elaborate paper on "Traffic Census," and what was to be gained by it, a State engineer said, "Well, after all this traffic census, you have got to use your common sense." That is true. That is the very object of having the traffic census, so that common sense may be used intelligently.

Now, the traffic is not the only thing that must be studied. One must study where the road is to be laid, what its objects are, the character of the different traffics, and, another thing, not only the entire traffic, but how that traffic is to be applied.

The engineer of the Borough of Fulham, London, has calculated that, under a certain traffic which has been observed, a wood pavement will last for a certain term of years; and that, when a wood pavement is laid on any street, he can calculate its life in accordance with the traffic which is to be applied. That must be considered to a great extent in a way which the speaker will mention later.

Now, after one understands fully the requirements of the road, the next thing is to find out the character of the materials available for use. This part of the question was not nearly as important 30 or 40 years ago as it is to-day. At that time, stone in its various forms was practically the only material used in road or street construction. Now, there is not only stone, but brick, wood, and the bituminous

Mr. Tillson. materials, the latter laid in all their various forms and composed of the different kinds of bitumens.

The engineer, in studying this phase, must consider first the character of the stone. He must know what the different kinds of stone will stand under the different kinds of traffic. The speaker was severely criticized once in college when he said—in considering the abstract question of foundations—that stone made the best foundations. His professor contended that the statement was absolutely wrong, because, under certain conditions, sand is just as good a foundation as stone. Now, under some conditions of traffic, a cheaper stone is just as good as a dearer, so that, though in a certain way one stone would be better abstractly, the other stone might be just as good concretely.

It is the same way with other materials. The engineer must understand brick and the different grades of brick; he must understand wood, what the different varieties of wood will do, what kind of treatment the wood requires; and then, when it comes to the bituminous pavements, he is uncertain at the present time. It does not make any difference where he goes, he will get different advice, different information. The information may be true, for the reason that different people are studying different kinds of bitumens, and they know more about the bitumens that they are studying than about any others, and while they are giving perfectly good and true information about their own, it seems to be contradictory to what was received before, because they did not understand the values of these other bitumens.

Now, let it be assumed that the engineer understands thoroughly the condition of his road, and all the different qualities of the materials, both in their crude state, and also how they can be mixed in order to produce satisfactory results.

The next thing is to make the application, and here there should be elaborated what was said about the traffic and the wearing out of the pavement in a certain length of time. Take, for instance, the assumption that a traffic of 60 000 000 tons will wear out a pavement, and the road will receive 2 000 000 tons of traffic per year. The logical deduction would be, then, that the pavement would last 30 years. Not necessarily; because there are certain materials in use which are affected chemically by climate and by the natural action of the atmosphere.

Now, as was stated by Mr. Green, wood and the bituminous pavements do require a certain amount of traffic in order to give the best results. So that, even if there exists a traffic which, theoretically and mathematically, would wear out the pavement in 30 years, this traffic may be so scattered that it would not be such as would be most economical and best for the pavement; and, on account of climatic conditions, it might be destroyed by disintegration and decay before it was worn out. Brick and stone are probably the only paving materials

now in use the life of which can be determined according to the foregoing principles, unless the traffic is so heavy that it will wear out the pavement before the materials decay or become disintegrated. The best illustration of this is in the use of wood. If these 60 000 000 tons of traffic will be applied to a pavement before the wood would naturally decay, there is no necessity for treating the wood with any material that will prevent it from decaying. That principle is recognized in Europe, where, in the great majority of cases, the wood will wear out before it will rot out, and it is only superficially treated. In Paris, where they use soft wood, and where the wear of the traffic in certain streets is such that the pavement will last only 7 or 8 years, they use what might be called no treatment; that is, the blocks are immersed for perhaps ½ hour in creosote oil and then laid in the streets. Mr. Tillson.

Paris, last summer, was erecting a plant in which the blocks will receive a treatment of 10 or 12 lb. per cu. ft. That is not necessarily to keep the blocks from decaying, but to prevent them from swelling and causing the pavement to bulge. That is the main thing that they hope to gain by increasing the quantity of preservative to 10 or 12 lb.

It can readily be seen that it would be wrong to use an elaborate form of treatment to prevent decay when the block itself will wear out before it would rot out.

In America, where there is not this intensive traffic, and where a pavement is expected to last longer, it is an entirely different proposition. Take a certain street, for instance, in Brooklyn, where wood block has been laid about 10 years. Practically no repairs have been made, and if the traffic continues as it has in the past 10 years—and being a residential street, it probably will—that pavement will not be worn out in 50 years from the time it was laid. So it will be seen how necessary it is to lay treated blocks under such traffic, so that they will last as long as they can, as far as decay is concerned; and for that reason it makes a difference whether the amount of traffic that will wear out a pavement is scattered over 4 or 5 years or 25 or 30 years.

There is just one little point more, which was referred to by Mr. Green and also to a certain extent, by Mr. Whinery. All these different principles that have been spoken of can be settled in the laboratory; but, if an engineer determines on granite, for instance, for a certain street, and writes to his client, who may be 400 or 500 miles away, and tells him of his decision, the client might write back and say there is not a granite quarry within 1 000 miles. The point to be recognized is the question of availability. That is of the utmost importance, and the speaker has often thought that, in the possibility, or rather the power, of Nature to furnish everything necessary for the development of the world, the development of civilization has been shown in the pavement line as much as in any other way.

Mr. Tillson. In the Central West, where there is no rock suitable for pavement, where all outside materials can only be brought in at great expense, Nature has furnished a material—clay—from which can be, and is, manufactured a brick that gives satisfaction and is almost as durable as the best of stone itself.

In summation, let it be said that if an engineer, who has roads or street pavements to plan, has first a knowledge of the conditions which his pavements must meet, if he understands thoroughly the qualities of the available materials, and if with that knowledge he makes his application in accordance with sound common sense, taking into consideration the atmospheric conditions and the availability of materials, he will undoubtedly reach a conclusion that will give satisfactory results.

The following question has been asked: "What conditions would be considered, and would govern, in making a selection of asphalt blocks, brick, or bituminous concrete for a street, say, in Brooklyn?" This is easily answered, because, as a matter of fact, Brooklyn uses only one of the materials mentioned, *viz.*, asphalt blocks. Brick has not been used because it is expensive and makes a more noisy pavement than asphalt, so that the latter is more desired by the property owners. Asphalt blocks are generally selected in preference to asphalt where the grades are comparatively steep. The blocks are less slippery than sheet-asphalt, as they are made of coarser materials, and on account of the joints between the blocks. Brick would be selected for a street on which the traffic would be too heavy for asphalt block. The speaker has never used asphaltic concrete, and cannot give an opinion as to whether this material would give a better wearing surface than asphalt blocks.

Mr. Meeker. ROBERT A. MEEKER, ESQ.*—The speaker cannot accept the theory of harmonic waves, said to be due to traffic, nor the beautiful system of harmony of the spheres, for, from practical experience, it has been learned in his Department that it is very necessary indeed to watch the output of the mixer carefully. The proportions of stone, of sand, and of asphalt may be all right, but the first run of the mixer will be too rich in asphalt, the middle of the run will be just right, and the latter part of the run will be too lean; consequently, one would have a pavement composed of, for example, 12%, 8%, and 4% of asphalt, all from the same mixer, notwithstanding that the stone was weighed and measured carefully, as was the asphalt, before either was put in the mixer.

This mechanical failure is often overlooked. It caused so much trouble that the speaker was led to make a careful investigation of the run of the mixer, and it was found that, in many cases, the asphalt concrete which had been condemned as not being properly mixed,

* State Highway Engineer of New Jersey.

owed its lack of uniformity, not to the workmen who were in charge (they had done their duty faithfully and carefully), but to the machine. Mr. Meeker.

In connection with this theory of harmonic waves, there was one very important point made by Mr. Washington which is worthy of more than passing notice, namely, the irregularities caused by improper rolling. The speaker has had considerable trouble in the past, even with water-bound macadam roads. When the causes of the so-called waves are carefully analyzed, it will be found that the trouble lies, not in the traffic, but in faulty construction. The first macadam roads built in the United States were rolled with heavy three-wheeled rollers. There never were any waves in those roads. Later, an endeavor was made to reduce the cost of construction, and, to that end, horse rollers were substituted for the heavy English steam rollers; still later, different types of steam rollers were used, and, last of all, the tandem roller was brought forward as the best. The speaker tried them, and had macadam roads with harmonic waves long before automobiles were used. The real reason for these waves, or hills and hollows, in the road, may be found in the method of construction. If the base is not thoroughly compacted, not thoroughly rolled, and not brought to an even and uniform surface, then, if those waves do not appear at once, they will develop eventually. When to this wavy base is applied bituminous concrete which is not uniform in composition, two sets of waves—two sets of hills and hollows—will appear, not due to traffic, but to careless construction.

Highway engineers should not try to dodge their responsibility for poor workmanship by charging it to motor vehicles or to the rhythm with which they travel over the highways, when the fault lies in the manner of construction. This must be carefully watched. Careless construction was one of the reasons for the failure of so many penetration roads. The speaker has had experience with penetration roads in which he was told that the distributing wagon had stood for a time at a certain point and that the hump of bitumen, seen on top of the road, was due to the fact that the controlling valves were leaking.

In one case, coming under the speaker's immediate observation, he knew that the wagon had not stopped within 100 ft. of that particular point; therefore, this explanation was evidently wrong; hence, a series of investigations was made, whereupon it was found that the percentage of voids in the road varied greatly. Where the road was comparatively dense, the bitumen appeared in humps on top of it and disappeared entirely at a depth of 2 in., and where hollows appeared it was found that the bitumen had penetrated in many cases to the depth of 7 in. The same quantity of bitumen per square yard of surface having been applied, there was consequently a recurrence of the fat and lean

Mr. Meeker. bituminous mixture encountered in the output of the mechanical mixer. A careful analysis of numerous faulty spots, in both broken-stone and bituminous pavements, has convinced the speaker that the varying percentage of voids is a fruitful cause of many so-called failures; in other words, a lack of homogeneity in the mass is the real cause of the wavy surfaces so often seen.

Mr. Beaty. R. E. Beaty, Assoc. M. Am. Soc. C. E.—These remarks may not have a literal application to the discussion of "Factors limiting the selection of materials and of methods in highway construction", but, with regard to the use of wood paving blocks, the speaker is of the opinion that some mistaken ideas are abroad which should be corrected.

In European practice it has been customary to use a very light treatment in preparing blocks, that is, to treat them with 10 lb. of oil per cu. ft., or in some instances merely to dip them into the preservative.

The wood principally used in France is known as "Landes" pine; in England "Baltic deal" is used. An examination of some of these blocks shows very uniform wear, and that the wood very much resembles southern short-leaf, yellow pine, with the difference that the wood is much more nearly uniform in density and strength of fiber.

Tests made by the Borough of Manhattan, and by the Department of Forestry, of the United States Department of Agriculture, show very clearly that although some American pines very much resemble the timber used in France and England, they are of much less uniform quality. The characteristics of southern pines will be considered in more detail later.

In the speaker's opinion, it will be taking a grave risk to try to follow the European practice of using a light treatment of oil. This method will have the effect of leaving some of the blocks in the pavement untreated. This condition is indicated to some extent in foreign countries, where numbers of blocks have to be removed on account of rotting.

All the speaker's observations of experiments and—what is of more value—of the actual condition of 250 000 sq. yd. of blocks after treatment, in charges of from 400 to 500 sq. yd., has convinced him that, as the quantity of oil is decreased below 20 lb. per cu. ft., it becomes increasingly difficult to get either a uniform distribution of oil between the individual blocks making up the charge, or a satisfactory penetration into all the blocks. This difficulty can be overcome to some slight extent by the slow application of the oil, but this is not commercially desirable, for the reason that lengthening the time of treatment would often curtail the output of plants fully 33 per cent. Attempts at light treatment have been observed many times. In

one case, 16 000 sq. yd. were being treated with 16 lb. per cu. ft. The specific gravity of the oil used was 1.06 at 38° cent. At first a 16-lb. treatment was tried, but it was found necessary to increase this to 18 lb., because many of the blocks, when the lighter treatment was applied, showed a penetration of only $\frac{1}{4}$ in. from the surface. The inference is, that in injecting 16 lb. or less into the blocks, the duration of time the pressure is applied to the treatment cylinder is too short to allow the pressure to be raised to a height sufficient to cause oil to penetrate the denser and harder blocks before all the oil is used up. In experiments reported in December, 1912, on various samples tested for the Department of Public Works, Manhattan, it was shown that the quantity of oil taken up per cubic foot by the blocks, under a pressure of 150 lb. per sq. in., which pressure was necessary to inject an average of 20 lb., varied widely among the different species of southern yellow pine blocks making up the samples. Some blocks received the equivalent of 28 lb. per cu. ft., and others received only 12 lb. At less than 50 lb. cylinder pressure, 10 lb. can be injected. If thus treated, many blocks in a charge would receive practically no oil. This lack of uniformity in material, though not so marked in the "Landes" pine and "Baltic fir", indicates a cause for decay in individual blocks. Mr. Beaty.

There is in very general use, by some wood block manufacturers, the term "Southern Yellow Pine" to describe the timber used in manufacturing their product. This nomenclature, as used in the 1913 *Proceedings* of the Association for Standardizing Paving Specifications, is intended to cover all the five or more varieties of pine found in the South, and it was so stated in the "discussion" at the time of their adoption. That the term is misleading, indefinite, and inaccurate is readily seen by a reference to Bulletin No. 13 of the Division of Forestry, of the United States Department of Agriculture, in which five species of pine, several of them occupying large and definite areas of the Southern States, are given, as follows:

"Long-leaf" pine (*"Pinus palustris"*, Miller);
"Cuban" pine (*"Pinus heterophylla"*);
"Short-leaf" pine (*"Pinus Echinata"*, Miller);
"Loblolly" pine (*"Pinus taeda"*, Linn); and
"Spruce" pine (*"Pinus glabia"*, Walt).

These species vary widely in physical characteristics, such as average percentage of resin, number of annual rings, weight per cubic foot, average percentage of heart wood, strength in compression of fiber parallel to the grain, etc. Two of these definite groups will be compared: the "Long-leaf" and the "Short-leaf" yellow pine. Tests

Mr. Beaty. by the U. S. Department of Agriculture showed the following relative values:

	Long-leaf.	Short-leaf.
Strength in cross-breaking......	10 900 lb.	9 230 lb.
Strength in compression (per square inch):		
Average lowest............	5 650 "	4 800 "
Average highest...........	6 850 "	5 900 "

In summarizing the results of tests on various pines the Department expert states: "From these results, though slightly at variance, we are justified in concluding that Cuban and Long-leaf are nearly alike in strength and weight, and excel Loblolly and Short-leaf by about 20 per cent."

These values were checked by experimental tests, at Columbia University, on samples secured by the speaker for the Borough of Manhattan, to which reference has previously been made. The results of these tests are not given here as the tests were made to demonstrate the value of a requirement for a minimum number of annual rings in timber used in the manufacture of blocks. It is interesting to note, however, that the average strength of three samples of long-leaf was more than 10 000 lb. per sq. in. in compression, parallel to the fiber, and some samples of short-leaf were as low as 5 600 lb. per sq. in.

Other tests, made at this time by the Highway Department, indicated clearly that "short-leaf" pine, as compared with "long-leaf", has a greater range of expansion and contraction, and is more difficult to impregnate uniformly with preservative. Each of the species considered has its uses, but, as they often vary widely in price, as well as in physical characteristics (as has been shown), then, for the mutual protection of the bidder and the ultimate user, engineers should specify clearly the kind of timber to be supplied.

In the matter of the limitation mentioned by Mr. Tillson, the speaker heartily concurs. It would not be reasonable to conclude that, simply because "a block might possibly wear out before it would rot out", it would be wise to use a very light treatment.

It would be necessary to use a certain quantity of oil in order to protect the blocks against bulging or "buckling". The proposition to use untreated timber is also unwise, because it has been observed that, if certain varieties of pine, if untreated, are exposed to the weather, they will be destroyed by fungi and other destructive agents in less than 4 years.

Mr. Meeker's remarks with regard to proper proportioning and mixing are extremely important. Several engineers of the Bureau of Highways, Manhattan, have endeavored for the past 2 years to restrict the use of machine mixers to those of the "batch" type. It

Mr. Beaty.

has been noted that many "continuous" mixers furnish a mix which is often too "lean", and at other times practically devoid of stone.

If continuous mixers are not considered safe for use on such structures as the Panama Canal, the Keokuk Dam, the New York Subway and the Aqueduct work, their use should not be permitted for mixing concrete for use in foundation for pavements designed for heavy traffic.

Mr. Kinney.

WILLIAM M. KINNEY, JUN. AM. SOC. C. E.—Referring to concrete pavements in which there appears to be some pulverizing action under the bituminous carpet, the speaker has made quite a number of investigations of pavements of this type and, in the few cases where such condition exists, has found that the surface of the pavement, prior to the application of the bituminous carpet, was of poor quality, either because of the use of poor aggregate, or because the surface of the concrete had been frosted.

In floating concrete made from an aggregate in which there is an objectionable quantity of dirt or loam, this unsatisfactory fine material is brought to the top of the concrete and makes a surface which is of poor strength and to which another material does not readily bond. A skin on the concrete thus formed might show the pulverizing action to which Mr. Crosby has referred.

When the concrete has been frosted prior to the application of the bituminous top, practically the same result would be expected, as the thin layer loosened from the concrete by the freezing action would pulverize rapidly under travel.

When properly made concrete is covered with a bituminous carpet, and this carpet later peels off, there is usually very little material such as sand, gravel, or mortar clinging to the bituminous material.

It would be interesting to know the location of the roads to which Mr. Crosby refers, and to ascertain whether a report on the character of the aggregate and time of doing the work is available.

Mr. Goldsmith.

WILLIAM GOLDSMITH, ASSOC. M. AM. SOC. C. E.—Speaking about harmonics and what Mr. Washington has said, the speaker would like to relate one little incident which happened on Second Avenue, in New York City, which brings out that point rather clearly.

From 1st to 23d Streets, on Second Avenue, experimental pavements have been laid and on one stretch, between 15th and 17th Streets, the pavement is of asphalt block. This section is in itself an experiment, different qualities of blocks being used within the area. One part of it is laid with soft blocks having a large percentage of bitumen. Other sections are laid with harder blocks.

About a year after the pavement was opened to traffic, a number of depressions and holes appeared, and the wave effect which has been mentioned was also evident. It became necessary to replace these

Mr. Goldsmith. defective parts, and in ripping up the old blocks, a curious condition was found. The asphalt blocks which were lined up perpendicular to the curb had taken a semicircular form; they had moved up on one another under the heavy traffic, so that waves were formed. Originally, the blocks were 5 in. wide, 12 in. long, and 3 in. deep. When removed their depth was found to vary from a minimum of 1 in. to a maximum of about 6½ in. The width and length had correspondingly increased or diminished, showing that there was little if any wear of the material, but that a distortion of the blocks had taken place. This showed conclusively that the asphalt moved or crept up and worked into semicircular shape, because the blocks were soft, and not on account of rolling. Adjoining these particular blocks were some of a harder nature which retained their shape, thickness, and alignment.

Approximately every 6 ft., and perpendicular to the curb, steel plates were embedded in the concrete foundation, extending up into the asphalt block joints. The object of these plates was to prevent the creeping and distortion of the blocks, which was anticipated. The plates, however, took the same semicircular shape with the blocks, the pressure evidently being great enough to detach them from the concrete in which they were embedded.

There was no grade on that part of the street where these blocks were laid. The traffic, both light and heavy, moved in the direction of the curves, and the latter had a middle ordinate of about 7 ft., the chord being about 20 ft.

This incident seems to show that in this case the wavy effect was due to the softness of the material under heavy traffic, and not to any other cause.

There is another point, which has not been spoken of, but is of great importance. It is the factors which limit the selection of pavement to be used in the railroad area in city streets—asphalt, wood block, granite, or other material used in city streets. Many streets have car tracks in the center. The question always arises, shall the same pavement that is used in the roadway be placed within the railroad area?

According to the usual railroad laws and charter requirements, the pavement between the railroad tracks and for 2 ft. on each side of them, is maintained by the railroad company, but must be constructed in accordance with specifications prepared by the city engineer.

Investigations in New York City have shown more depressions and other defects on pavements between the tracks and near them, than at other places. The question, therefore, naturally arises, which is the best pavement to place in the railroad area? Asphalt, wood block, or granite?

The consensus of opinion in Manhattan Borough seems to be that the improved granite block is the most satisfactory. It has some dis-

advantages, however, on account of the ties which come in between the slot rail and track, which make it necessary to chip the granite blocks to make them fit well between these cross-ties. Mr. Goldsmith.

The joints, in Manhattan Borough, are made with paving pitch and sand. The speaker is informed that the street railroad companies in Brooklyn have used cement grout almost entirely during the past 8 or 10 years, and very successfully.

This subject is brought up with the hope that some one will discuss it and give his experience with relation to pavements in the railroad area.

GEORGE A. RICKER, M. AM. SOC. C. E.—In reference to this subject of materials some curious things have happened in the New York State Highway Department, as disclosed by recent investigations, and steps are being taken to try and correct mistakes that have been made. Mr. Ricker.

Roads were designed by some of the engineers who had never seen the territory in which the road was located. Specifications were applied to sections of which the engineers were totally ignorant of conditions. The speaker found places where solid rock was being excavated, and the road replacing it made of concrete, and where bituminous roads were built on blow sand foundations. At present the Division Engineers are required to send in, with their plans of roads to be improved, a report of their personal inspection after walking over them.

The Department has also instituted a survey of the State to locate available materials. One of the Assistant Engineers, who is somewhat of a geologist, and has been engaged in the Testing Bureau for some years, has been assigned to the work of examining materials in all parts of the State, and a chart is being compiled as a record of his examination.

D. B. GOODSELL, ASSOC. M. AM. SOC. C. E.—The speaker cannot say much about "harmonic waves" in asphalt or bituminous pavements, but readily agrees with what Mr. Washington has said regarding Mr. Crompton's ideas as to rolling. He believes that waves such as mentioned by Mr. Crompton might occur in some hard homogeneous material, but, with one as plastic as a bituminous pavement, such waves are likely to be due to two causes: lack of cross-rolling, and lack of homogeneity of the material. Most highway engineers are familiar with the effect of cross-rolling, and the speaker believes that, if roads generally were rolled crossways, as well as longitudinally, the regular waves would not occur as frequently. Mr. Goodsell.

No doubt the uniformity of the material, or in other words, the number of voids in it, has a very great influence on the evenness of wear; and the secret of obtaining a good road, other things being equal, seems to be the even distribution of voids throughout the material.

Mr. Goodsell. Attention is called to the measurements of wear of pitch-macadam pavement made by Mr. John Brodie, City Engineer of Liverpool, which seems to be the only instance of systematic measurement of wear under known tonnage of traffic. Some method such as he uses seems to be desirable for the determination of the relative life of the various kinds of hard pavement, including asphalt block. The speaker is not informed of any measurements which show the relative wear of asphalt-block pavements, and believes that this subject is an interesting field for investigation.

Mr. Durham. H. W. DURHAM, M. AM. SOC. C. E.—Reference has been made to stone block pavements in the car tracks, and to the different methods of filling joints. The speaker thinks that one important point in New York City practice was not brought out as clearly as possible. It explains very largely the difference between the practice in the Boroughs of Brooklyn and The Bronx and that adopted for Manhattan, and is mostly due to the radical difference in the types of track construction. In Manhattan an underground current-conductor with central slot between the rails is used almost entirely. In all the other Boroughs current is taken from an overhead wire, necessitating no special work in the track. The Manhattan construction requires the possibility of very frequent access to the substructure, for repairs either to the current-conductor system or the cast-iron yokes carrying the track. Therefore, it has been found more desirable to use a type of pavement which can be easily removed and promptly restored after repairs are made.

The use of a cement grout as a filler gives an excellent surface between the rails, as may be seen on some of the Brooklyn and Bronx streets; but if this were used in the Manhattan track construction it would render very difficult the making of any openings for repairs to the numerous hand and manhole boxes and other sub-surface structures, and it would also be difficult to obtain proper setting of the grout filler after repairs were made. This largely explains the difference in practice.

EQUIPMENT AND METHODS OF MAINTAINING BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

Mr. Spencer. HERBERT SPENCER, ASSOC. M. AM. SOC. C. E.—Any discussion of a subject such as the equipment and methods for maintaining bituminous surfaces and bituminous pavements must, of necessity, be limited by the length of time such surfaces have been in existence, and should properly include: (1) classification of the surfaces and pavements required to be maintained; (2) methods and material used in their construction, together with the causes responsible for their failure; and (3) equipment and methods required for their maintenance. The subject being of such importance, and having such possibilities of detail,

the writer is of the opinion that a summary of these headings is all that should properly be attempted in a limited discussion of this nature. It must also be remembered that bituminous surfaces and pavements of various kinds having a large mileage, have ended their period of usefulness, and their maintenance now becomes a matter of first importance and requires the development of machinery and methods for their economical upkeep. These can come only from actual experience, and the lines along which such development should follow is all that can be attempted at present. Mr. Spencer.

During the past 5 years many types of bituminous surfaces and pavements have been placed on the public streets and highways. Some of these have attained sufficient merit in the public estimation to warrant their continuation, and others have shown such faults in their original design and construction that their further development was not considered economical or wise. Without doubt, new types of bituminous surfaces and pavements will be exploited with the growth of highway work, but it is felt that these cannot deviate very much from the existing types, which are defined by the Special Committee on Bituminous Materials for Road Construction as consisting of superficial coats of bituminous material with or without the addition of stone or slag chips, gravel, sand, or material of similar character. This definition may include: (*a*) treatment of gravel or stone roads with cold bituminous material; (*b*) treatment with hot bituminous material; or (*c*) treatment of a Portland cement concrete base with either hot or cold bituminous material.

The Special Committee on Bituminous Materials for Road Construction defines a bituminous pavement as "One composed of stone, gravel, sand, shell, or slag, or combinations thereof, and bituminous materials incorporated together by mixing methods." This definition may include: bituminous concrete, laid either hot or cold; bituminous macadam (penetration method); bituminous mortars, consisting of bituminous cement and fine material aggregate, such as rock asphalt, and laid either hot or cold; sheet-asphalt, asphalt block, and similar pavements

The maintenance of bituminous surfaces and bituminous pavements is measured largely by the methods and materials used in their original construction. Although it is not considered necessary to enter into a discussion here of the specifications for any of the types just mentioned, the generally accepted limitations of methods of construction may be summarized as an index to the extent of maintenance.

For bituminous surfaces, the thorough sweeping of the base is of great importance for applications of hot bituminous materials, and should be resorted to for cold applications wherever an extremely dusty condition of the surface is found. The bituminous materials used range from asphalt oils and tars to the heavier products, of like char-

Mr. Spencer.

acteristics, requiring heat for their application. Of importance is the selection of the cover. For the hot applications, coarse sand or grits, preferably washed, are commonly used for automobile traffic; but, for steep grades and horse-drawn traffic, stone screenings are better. The items, therefore, influencing the success or failure of bituminous surfaces include the preparation of the base, the selection of the bituminous material, the selection of the cover, and, of equal importance, the machinery to be used in the application of the bituminous material. The treatment of cement concrete surfaces with bituminous material is of too recent origin to warrant an opinion of probable causes of failure, but may be due to any of the following: (1) base floated to a smooth surface, precluding possibility of any bond with bituminous material; (2) bituminous material lacks necessary characteristics to enable it to adhere to concrete; (3) base too hard and surface too thin to resist impact of horse-drawn traffic and raveling of surface results.

The danger of failure of bituminous pavements is lessened when care is used in the preparation of specifications and intelligence is shown in the inspection of the work. When failures do occur, they can generally be traced to unsuitable materials, poor foundations, improper machinery, or inferior workmanship. Bituminous concrete mixtures depend to a large extent on their structural stability, which is determined by their density, and by the continuous adhesion of the bitumen-covered particles under traffic, and violation of such well-known principles as grading of the mineral aggregate, consistency of bitumen used, or temperature of mixture and manner of laying, will lead to failure. For bituminous macadam pavements constructed under the penetration method, the quality of the stone, the proper sizing of the stone for each successive course, and the consistency of the bitumen used often determine the success of the pavement. These features were discussed by the writer in the 1912 Road Meeting of the Society.* It is not considered advisable to enter into a description of the causes of failure of sheet-asphalt, asphalt block, and similar pavements at this time, as this subject is fully covered in the discussions of Societies giving particular attention to these classes of pavements.

Bituminous surfaces originally treated with cold bituminous material may be renewed at slight cost. The extent of sweeping such surfaces is governed by the area and character of the existing cover on the road. If all loose dirt is swept from the original surface, and the bituminous material is covered with clean sand or screenings of a character not readily ground up by the traffic, then a re-treatment without the necessity of sweeping may safely be attempted. Roads properly cleaned, oiled, and sanded should leave a coating of bituminous material

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 615.

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to which the fresh material can readily attach itself. The quantity need not exceed $\frac{3}{8}$ gal. per sq. yd., and if sand is required, not more than 10 lb. per sq. yd. should be used, or a mushy, soft condition will follow. The bituminous material should be of such consistency that it can be readily applied by a pressure distributor, and should contain a minimum of material likely to lubricate the stone base. Where more than one application per season is required, the road should be treated in the fall, in order to assist as much as possible in maintaining the surface through the winter.

The most successful hot oil or blanket treatment, in the writer's opinion, is that which has been done in Massachusetts; and, on many miles of roads which he has been over in that State the problem of maintenance has been solved successfully. Mr. Farrington has carefully gone into the methods of maintaining this type of surfacing, and detailed descriptions, extending over a period of 4 years and given in previous publications of this Society by Messrs. A. W. Dean, F. C. Pillsbury, and J. A. Johnston, should serve as a model in those communities which really desire economical maintenance on roads subjected to heavy automobile traffic. For roads treated by the hot blanket method and showing the holes due to settlement of foundation, heaving by frost action, or wearing away, caused by lack of adherence to the base, the earlier such defects are repaired the longer will be the life of the treatment. After 2 years' service of a treatment of this nature, during which time not more than one-third of the surface should be in need of repairs, it becomes a question of the advisability of re-treating the entire surface or of tearing it up and resurfacing with a more permanent pavement. For the repair of minor holes, a well organized gang can readily keep a stretch of road in perfect condition at a moderate outlay for plant and materials. Such a repair outfit should have a portable kettle, for heating the bituminous material, and a compartment for keeping dry the grits or screenings. The hole to be repaired should be swept clean and painted with hot bituminous material of the same consistency as that used in the original treatment, and then filled with grits or screenings to the level of the surrounding surface. This should then be grouted with bituminous material and covered with a light layer of stone chips or gravel. Where practicable, the patches should be rolled, and should be carefully watched to see that the traffic does not tear them out before they properly set. Another method of repairing such bituminous surfaces successfully is by mixing the aggregate and bituminous material away from the site of the road and depositing them in the hole to be repaired. This should be tamped, sanded, and thrown open to traffic. The writer believes this method leads to more uniform results.

Bituminous Pavements.—The maintenance of the older types of bituminous pavements has been sufficiently studied to need no elabora-

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tion here. This refers to bituminous concrete, sheet-asphalt, asphalt block, and rock asphalt pavements. The more recent bituminous macadam pavements constructed under the penetration method in many cases show the need of immediate repairs. A study of the causes responsible for such failure should be made with a view of obviating these defects in the original construction. The writer, however, does not at all agree that the penetration method is the total failure that some authorities would have us believe, but is convinced, on the contrary, that for certain kinds of traffic this type of pavement is good enough. This conviction is based on the features of low first cost, length of service, ease of repair, and moderate charge for plant rental with which to apply the bituminous materials properly. Penetration pavements have now been down for a period extending through five summers and winters, have had no repairs spent on them, and are still in perfect condition. Where failures have occurred they can generally be traced to some defect in the original construction. The main causes of such failures are: (1) improper foundation; (2) improper sizes of mineral aggregate; (3) sealing of top course before applying bituminous material; (4) bituminous material of wrong consistency; and (5) uneven application of the materials. The various ways that failures occur in work of this nature are shown on the surface by: (*a*) wavy condition, showing either an unequal distribution of bituminous material or lack of bond to the underlying stone; (*b*) depressions in the surface, due to settlement of the foundation; (*c*) raveling, due to the bituminous material not adhering to the stone; (*d*) surplus of bituminous material on the surface, due to the difficulty of penetrating the stone, combined with improper consistency. For patching holes in a penetration road it is best to clean it thoroughly, scarify the surface, and add sufficient stone to bring the top course to the level of the surrounding road. The heated bituminous material is then applied, and stone of the next smaller size is rammed into the interstices. Generally, only one application is required, and care should be taken that the patch is at the same level as the surface of the road. Where there is an extreme wavy condition, it would probably be found more advisable to scarify the entire surface, adding sufficient stone, if necessary, and treating it with a light application of bituminous material followed with screenings and thorough rolling. Where raveling has taken place, a blanket treatment of bituminous material, applied under pressure, will often seal the road, without the necessity of scarifying.

Bituminous concrete pavements laid cold (including bituminous mortar) are ordinarily maintained with material prepared at some central point and shipped to its destination. This may be used to repair holes or to spread over the surface to form a new roadway.

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Equipment.—For the application of bituminous material, by far the most successful method is by the pressure distributor. This may be horse-drawn, operating a small pump by a sprocket chain, or the pump may be driven by a gas engine in the rear of the wagon; but the most economical is the auto-driven sprayer. The speed can be regulated to spread any quantity required in a uniform spray, and under uniform pressure. For the blanket treatment, there has recently been placed on the market a machine designed to apply heated materials under pressure. This is considered the most advanced and improved method of applying heated material, and its use should be encouraged wherever possible. The outfit consists of a double-shell, asbestos-lined tank, holding from 750 to 1 000 gal., and containing 1½-in. heater pipes. A boiler, equipped to burn either coal or fuel oil, is carried on the rear. This furnishes steam to heat the bituminous material in the tank and to operate an air compressor. The bituminous material is forced through the outlet nozzles under the pressure desired, the quantity being gauged by the speed of the truck and the size of the outlets. This machine can apply the heavier material used in the construction and maintenance of bituminous pavements under the penetration method, and can readily apply the seal coat on bituminous concrete pavements. Where long stretches of road are in need of repairs, the auto-truck sprayer will be found the most economical, and where communities do not care to invest in the costly machines of this nature their services can be contracted for, on a gallon or yardage basis, from the companies operating them.

Indirectly related to the question of maintenance is the uniformity of the bituminous material to be used. There have been discussions at various times relating to the physical and chemical requirements of bituminous materials, and specifications, designed to limit and control their manufacture, have been drawn. Speaking only in reference to petroleum asphalts, with which the writer is more familiar, a few of the features governing the manufacture of the products under given specifications will be cited as illustrating some of the difficulties attending its preparation. In the first place, for a given grade of crude oil, a uniform product should result from the manufacturers' plant. On the other hand, elaborate tests may be interesting, as a matter of record and to classify the asphalt used in any given piece of construction, but when the manufacturer is asked to meet a variety of tests, necessitating a change in his process, a hardship is imposed on him, with no betterment of the resulting product.

The manufacturer who uses stills in the preparation of his material is governed largely by three fundamental conditions: (*a*) the character of the base material he is refining; (*b*) the temperature of distillation; and (*c*) the time of distillation. For a given crude supply, which may be considered satisfactory for the preparation of asphalt, it will be

Mr. Spencer. seen that the resulting product is dependent largely on the temperature and the time of distillation. In many cases any change in one to meet a given test will affect seriously more important and necessary tests. As an illustration may be mentioned the fixed carbon requirement in certain specifications. It is agreed, on excellent authority, that the fixed carbon in an asphalt is the product found by dividing the fixed carbon in the crude material by the yield. Therefore a blanket clause in specifications limiting the percentage of fixed carbon, without taking into consideration the crude material which is being run, imposes a condition which the manufacturer is powerless to meet. He cannot control the minimum of fixed carbon, which is governed entirely by the yield and percentage in the crude, and a certain leeway should be given him for the maximum, to allow for errors in the test, imperfections in the process, and causes over which he has no control. Temperatures may control this maximum; but, in making a product for the highest class of paving work, there is no incentive or desire on the part of the manufacturer to injure the product in any way possible, and he would have no object in running at a temperature other than that which will produce the best material.

Engineers are prone to adopt technical specifications for materials used by other communities without investigating their merits as applied to their own work. Although this may be taken as a compliment to the originators of such specifications, in many cases it is done through a lack of knowledge of the materials to be used, whereas the unbiased judgment of those qualified to speak, with an eye to the considerations of cost and durability, will aid many engineers in the intelligent preparation of specifications.

The opinions of engineers assembled to discuss almost any form of construction and tests on most grades of materials will represent the most up-to-date and best thought, and engineers cannot go far astray in adopting such specifications, provided they have been tried and in actual use. Therefore national societies have a duty to perform in perfecting specifications for paving materials which are to be used generally throughout the country, and care should be taken that such specifications contain only those items which will insure a satisfactory product for the work to be done and not impose conditions impossible to fulfill with the materials available.

Mr. Boorman. T. Hugh Boorman, Esq.—In the discussion 2 years ago the speaker stated that Nelson P. Lewis, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment of the City of New York, had considered that rock-asphalt pavements were probably the best that had been laid. George W. Tillson, M. Am. Soc. C. E., objected to it because he considered the maintenance charges on it excessive. The speaker, therefore, begs to state that rock-asphalt comprimé has been

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laid in the United States in New York, Brooklyn, Long Island City, and Rochester, in New York State; Elizabeth and Perth Amboy, in New Jersey; Boston, Mass.; New Haven, Conn.; Philadelphia, Pa.; and New Orleans, La.

The first comprimé work was laid in Union Square, New York, in 1872. In August, 1897, 112th Street, from Fifth to Lenox Avenues was laid, and the city records show that in 1912 the cost of repairs on this street was only 6 cents per sq. yd.; and 101st Street, from Lexington Avenue to Park Avenue, paved in July, 1896, with Mons and Sicilian rock, bears no cost for repairs from 1910 to 1913. Such durability seems to be unparalleled in the history of street construction. On Dyckman Street, from Kingsbridge Road to the tracks of the New York Central and Hudson River Railroad, 8 000 sq. yd. of Seyssel and Sicilian rock asphalt were laid in the late fall of 1897, and the cost of maintenance for the 4 years, 1910 to 1913, inclusive, was 3 cents per sq. yd., or less than 1 cent per sq. yd. per year. In 1901, between Park and Lexington Avenues, 35th and 36th Streets were paved with Seyssel and Sicilian rock asphalt; on 36th Street no repairs were necessary for the 4 years, 1910-13. In the same year 13th Street, from Second to Third Avenues, was paved with Sicilian and Mons rock on an old stone foundation, and is charged with 14 cents per sq. yd. for 4 years' maintenance. In 1897, 106th Street, from Broadway to Riverside Drive was paved with Sicilian and Mons rock, and shows cost for repairs of 17 cents per sq. yd. for 4 years, a trifle more than 4 cents per year.

To come to more recent times, it was decided in 1912 by the Department of Highways, under the advice of E. P. Goodrich, M. Am. Soc. C. E., Consulting Engineer to the President of the Borough of Manhattan, to lay a series of test pavements on Second Avenue, from Houston to 23d Streets. Rock asphalt comprimé, rock-asphalt blocks, and Sicilian rock comprimé were laid between 19th and 21st Streets, and on inspection, in January, 1914, these two blocks were found to be in good condition, the only imperfection being in the work adjoining the car tracks. This is not to be wondered at, as all rock asphalt experts, such as Malo, Delano, Walsh of Amsterdam, Bassett of London, and almost all engineers in Europe have decided that granite or other blocks should be laid longitudinally beside car tracks. The speaker recommends scoria blocks. The most noticeable of all the experimental pavements on Second Avenue is that of rock asphalt compressed blocks, between 9th and 11th Streets. These blocks were manufactured, at the Staten Island Works of the Sicilian Asphalt Paving Company, in an ordinary German brick machine, and did not receive the perfect compression obtained by the Val de Travers Asphalte Paving Company in their works at Marseilles, Seyssel, Cairo, Egypt, and other places. However, the rock being pure crude rock asphalt powder before being

Mr. Boorman. pressed, simply spread under heavy traffic, and became virtually a monolithic sheet of rock asphalt, and it affords to-day as perfect a specimen of rock asphalt pavement as can be seen in any country.

Mr. Delano, on the occasion of his visit to New York in November, 1913, stated that the City of Paris had recently signed contracts for replacing wood blocks, stone sets, and macadam with 700 000 sq. m. of rock asphalt pavement, to be laid during the next 5 years. Attention is also called to the fact that, some years ago, Thomas Street, from Broadway to Church Street, and Trimble Place, from Thomas to Duane Streets, both private streets owned by fifty associates, were paved with Neuchatel Rock Asphalte Pavés and coated with an asphalt rubber surface coat, only $\frac{3}{8}$ in. thick, which has stood the heavy traffic of those busy thoroughfares for several years.

The speaker can prove that rock asphalt is the most valuable pavement in the United States. This is stated especially because so many engineers doubt that any European rock asphalt, such as laid in London, Paris, and Berlin, has ever been laid in the United States. The speaker is sorry that, owing to the short time allotted to him, he will not have the opportunity to say a little about its history in America.

The reason this subject of the practicability of the use of European Rock Asphalte is so much more important than when the discussion before the Society was held in 1912, is that the tariff on asphalt has been removed, and Rock Asphalte powder can now be purchased at a cost of $3 per ton less than in 1913, making a difference of 25 cents per sq. yd. in the cost of the surface. Municipal plants can be supplied with the material, all ready for heating and compressing on the street, and no additional outlay for machinery is necessary. Both the Borough of Manhattan, which is now completing its new municipal asphalt plant, and the City of Philadelphia are considering the advisability of doing their own repairs to rock asphalt streets.

Mr. Sharples. PHILIP P. SHARPLES, ESQ.*—One or two points brought up in the discussion were not entered into very fully. For a number of years the speaker has been very much interested in studying the effect of oil on tar-bound macadam pavements. In considering the subject, country road practice and city or town practice must be sharply differentiated.

In building country roads, such as those described by Mr. Farrington, it has been the practice in many cases to construct a tar-bound macadam road, and coat it with one of the heavy asphaltic oils or light asphalts put on hot. This practice has given excellent results in many cases, and has been established as a standard specification in some States, notably in Connecticut. Some very good work has been done in this way.

* Chf. Chemist, Barrett Mfg. Co., Boston, Mass.

Mr. Sharples.

In carrying out work under a specification of this kind, it is very important that the grade of oil or oil asphalt be so heavy that it will not penetrate the tar-bound macadam, but will stay on top as a blanket layer. An oil light enough to penetrate the tar-bound road is sure to give bad results. The speaker knows of a number of roads of this type, which were well built and then treated with a light oil, while still presenting a porous top. They disintegrated completely within a year when subjected to heavy horse-drawn traffic, although neighboring sections sealed with tar products remained in good condition. If, however, a proper grade of asphaltic oil is used, this disintegration will not occur.

Under city or town traffic conditions, the tendency of an oil-asphalt top over a tar-bound macadam is to mush up and become soft under horse-drawn traffic in wet weather, due to the emulsifying effect of the water and organic material which collects on the asphalt top. When these conditions are to be provided for, it is important, so far as present experience indicates, to use only tar seal coats on top of the tar-bound macadam. The refined tars used for seal coats are preferably somewhat softer than the tar used in the tar-bound macadam, but the subsequent treatment and care of the road modifies somewhat the character of the seal coat required.

For maintenance within city and town limits, where the horse-drawn traffic is considerable, it has been found that a specially prepared tar, which can be applied cold, gives much better results than the heavier tar materials put on hot.

There are records of streets, in the vicinity of Boston, which carry more than 4 000 vehicles per day, which have been kept in good shape since 1908 with an annual maintenance cost of about 3 or 4 cents per sq. yd., with prepared tar applied cold.

Consider the light dust-laying oils and their effect on tar-bound macadam. It is a problem which affects city and town conditions. In country practice, the conditions do not arise where an oil of this type is necessary. On city and town streets, a troublesome dust often collects, which the ordinary seal coat of tar or heavy asphalt does not keep down. In applying a light oil over a tar-bound macadam to keep down this superficial dust, it is essential to comply with certain conditions if the tar-bound macadam is not to be ruined. It is the utmost folly to put a light oil on a tar-bound macadam; if there is the least chance for the oil to penetrate the macadam, the results are sure to be disastrous. The tar-bound macadam will be disintegrated. A light oil has two effects on the tar-bound macadam. It has a chemical effect on the tar which causes its disintegration and makes it lose its binding force. It also has a physical effect on the macadam structure. The oil is a lubricant; and if there is any stone loose enough to move in the tar macadam, the light oil accelerates the movement,

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and a hole is sure to develop. This dual action of the light oil can be observed in many large cities at the present time, and many streets may be seen which have been built with tar-bound macadam, the life of which has been gradually shortened by the use of light oil.

The effects of the light oil can be minimized if the tar-bound macadam is thoroughly sealed, either through an excess of bituminous material in the first place, or by subsequent applications of refined tar. Certain precautions must be taken also in applying the oil. An excess of oil should never be used, and it should be applied to the surface only in sufficient quantity to take up the loose dust which is found there. A very small fraction of a gallon is sufficient. If more than this is applied, the oil gradually sinks into the tar-bound macadam and in the end will rot it.

Wherever a light oil treatment is used on a city or town street for dust-laying purposes, it is quite essential to revivify the surface of a tar-bound macadam from time to time by the application of a tar seal coat. For this purpose nothing seems to be better than a prepared tar of such consistency that it can be applied cold. Only a very small quantity is required, usually not more than $\frac{1}{4}$ gal. per sq. yd. per annum. If these applications are put on intelligently, and if patching is intelligently attended to, a tar-bound macadam will give good service, even in the face of light oil applications.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

By L. J. Le Conte, M. Am. Soc. C. E.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much pleased with the results of the investigation given by the author. In May, 1901, he wrote a paper on this interesting subject for the American Water Works Association. This paper was intended only as a rough general guide to aid the judgment of engineers who were in trouble. Since that time he has been called on repeatedly to explain. Accordingly, he gives the following specific case: Mr. Le Conte.

Let x = average rainfall for the 3 dryest years on record.

The average run-off $= \frac{x}{2}$

The average catch $= \frac{x}{2} \times \frac{1}{4} = \frac{x}{8}$.

Assume x $= 40$ in.

Then average catch $= \frac{40}{8} = 5$ in. per annum.

5 × 17 400 000 gal. = 87 000 000 gal. per sq. mile; continued for 3 years = 260 000 000 gal. per sq. mile.

Where local experience indicates that it takes 2.5 sq. miles of water-shed to furnish 1 000 000 gal. per day in a long dry spell, then 260 × 2.5 = 650 000 000 gal. storage, which is necessary to insure a supply of 1 000 000 gal. per day in a long dry spell.

* Continued from February, 1914, *Proceedings*.

Mr. Le Conte.

Referring to Fig. 32, the writer is pleased to note a practical corroboration of this result. The Croton storage is given as 101 300 000 000 gal., and the water-shed as 360 sq. miles. This makes a storage of 290 000 000 gal. per sq. mile, for New York. The Spring Valley Reservoirs, at San Francisco, Cal., have 30 000 000 000 gal. storage for 30 sq. miles of water-shed, or, 1 000 000 000 gal. storage per sq. mile.

Then, $\frac{1\,000}{290} = 3.45$ times as much storage as is required in New York. Why? Simply because at San Francisco it takes 5 sq. miles. of water-shed to furnish 1 000 000 gal. per day, whereas in New York it only requires 1.5 sq. miles to deliver that quantity. In the New York case then, $260 \times 1.5 = 390\,000\,000$ gal. storage, and in the San Francisco case $260 \times 5. = 1\,300\,000\,000$ gal. storage.

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PAPERS AND DISCUSSIONS

REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.

Discussion.*

By Messrs. A. W. Buel, and Edward Wegmann.

A. W. Buel, M. Am. Soc. C. E. (by letter).—The author assumes the ratio of the coefficient of elasticity of steel and concrete at 10, which would seem to be low enough for any purpose, and sufficiently safe. Then, in the formula for the value of T, if the writer understands it correctly, the author seems to use a ratio of 9, making the tension in the steel, for 290 lb. per sq. in. in the concrete, only 2 610, instead of 2 900. Mr. Buel.

It would be interesting to know what the comparative cost would have been if, instead of proportioning the thickness of the concrete on the theory used by the author that vertical cracks would not develop if the tension be limited to 290 lb. per sq. in., the concrete had been made with a uniform and minimum thickness from top to bottom and with a separate water-proof lining inside. Unless there is a considerable saving in cost, the separate water-proof lining would appear to have the advantage of the other method, on the ground of many successful precedents.

Edward Wegmann, M. Am. Soc. C. E.—Although not directly a discussion of the paper under consideration which the speaker has been asked to discuss, it may be interesting to describe the recent partial failure of the reinforced concrete Ambursen dam, built in 1912 and 1913 across Stony River in West Virginia, for the West Virginia Pulp and Paper Company. Mr. Wegmann.

In the fall of 1911, this company engaged the speaker's services as Consulting Engineer to examine the site selected for its dam, report whether it was suitable for this purpose, and recommend the

* Continued from February, 1914, *Proceedings*.

Mr. Wegmann. type of dam that should be constructed. The dam was to be built on top of a mountain at the end of a log railroad. Owing to the great depth to rock, the cost of a masonry dam was prohibitory. There was no suitable material available for an earth dam, and therefore the speaker recommended that a hollow dam of reinforced concrete be built.

After a number of test pits had been dug and some borings had been made, four construction companies were invited to submit plans for the dam and bids for its construction. Before doing so, each of these companies sent a representative to inspect the site of the dam and the test pits. The plans submitted have been fully described.* Each construction company bid a lump sum for building the dam with a cut-off wall down to a certain assumed probable depth. Below this depth the cut-off wall was to be paid for by the cubic yard.

The dam was to be about 1 000 ft. long and was to have a maximum height of about 50 ft. above the surface. The lowest bid was $143 000, received from Mr. F. G. Webber. The bid of the Ambursen Hydraulic Construction Company, of Boston, was $189 000. In view of the fact that the latter company was the only one of the four bidders which had built a large number of reinforced concrete dams, the speaker recommended that the Ambursen Company be engaged to make the plans for the proposed dam. This was done. The contract for building the dam was awarded to Mr. Webber. To insure that the work would be properly done, the Paper Company engaged one of the engineers of the Ambursen Company to be constantly on the ground during the construction.

Mr. Webber began work at the west end of the dam and built about half the structure. The work was then taken away from him, as the progress he made was not satisfactory, and the dam was completed by the Ambursen Company at cost plus a certain percentage for profit.†

It appears that for about 200 ft. from the west end of the dam, the cut-off wall was founded on hard clay, and was made only 5 ft. deep below the floor of the dam. This shallow cut-off wall was built to where the water was from 30 to 35 ft. deep. For the remaining length of the dam, the cut-off wall was founded on rock. This part of the dam has remained intact.

The failure was caused by water finding its way under the cut-off wall, through a permeable, narrow seam of stone, sand, etc., that occurred in the clay formation. If the cut-off wall had been carried down a foot or two deeper, it would have intercepted this seam.

The water that passed through the seam showed itself in the weepers of the floor three or four days before the failure occurred. The

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* *Engineering News*, September 5th, 1912.

† The manner in which the dam was built is described in *Engineering News*, of January 22d, 1914, and in the same issue there is an account of the failure of about 75 lin. ft. of the dam, three or four months after the reservoir was filled.

watchman, left in charge of the dam, did not appreciate the danger this leakage indicated. It was only a day before the failure, that it occurred to him to inform the Superintendent of the Paper Mill at Luke, Md., about the leakage, which was steadily increasing in volume. Mr. Wegmann.

At that time a blizzard was raging and the temperature was 10° below zero. When the Superintendent arrived, the following day, the dam had been undermined and about 75 lin. ft. of the structure was destroyed.

The accounts of this failure which appeared in the daily newspapers were greatly exaggerated. The reservoir stored only about 800 000 000 gal. From the dam to the Potomac River, Stony River flows through a wilderness, the only building being a trapper's hut. The damage done, exclusive of the destruction of part of the dam, was very slight.

The failure was evidently due to the shallowness of the cut-off wall. In asking for lump bids for building the dam, the engineer of the Paper Company and the speaker had to assume certain depths to which the cut-off wall would probably be built. This depth was made 7 ft. below the top of the floor of the dam at the ends of the structure, 10 ft. where the depth of water was from 20 to 25 ft., and the wall was to be carried down to rock in the center of the valley. At each end of the dam sheet-piling was shown. The depth to which the cut-off wall was to be built had to be determined, of course, during the construction.

As actually built, the core-wall was founded on rock for about three-quarters of the length of the dam, and for the remaining length it was made only 5 ft. deep below the bottom of the floor of the dam. No sheet-piling was used.

The speaker's connection with the work ceased when the contract for building the dam was awarded, which was before the plans prepared by the Ambursen Hydraulic Construction Company had been received, and he had nothing to do with the construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

STEEL STRESSES IN FLAT SLABS.

Discussion.*

By Messrs. Edward Godfrey, H. E. Eckles, and Sanford E. Thompson.

Edward Godfrey, M. Am. Soc. C. E. (by letter).—The flat slab has repeatedly been brought before the Engineering Profession for consideration and adoption, on the plea that it is efficient and economical. Before adopting it, however, the Profession has a right to examine its credentials. The theory of the flat slab must pass muster; and tests on it must be consistent and rightly interpreted. Some things in reinforced concrete design have been adopted, which have not met these conditions. They were adopted in ignorance and have been the cause of unnumbered wrecks. One of these is the stirrup or short shear member, another is the rodded column with no lateral reinforcement; but these will be rooted out in time. That they ever received the sanction of the Profession is a disgrace. It will not do to add to the number of such ill-considered standards by the adoption of any other untried and illogical method of design. Mr. Godfrey.

The theory of flat slabs, as given by most writers, would tend to place it in the innocuous class, for the great thickness of slab demanded would put it "out of the running"; but the commercial designers succeed in scaling down the bending moments to a point which independent practising engineers fear to attempt.

The flat slab theory premises a slab of indefinite extent, supported on an indefinite number of columns, the entire slab being uniformly loaded. The term, cantilever flat slab, so often used, conveys the idea that over the column heads the slab is a cantilever. The premises and the several assumptions are faulty. The slab is not of indefinite extent.

* This discussion (of the paper by H. T. Eddy, Esq., published in January, 1914, *Proceedings*, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Godfrey.

It must end at the outer row of columns or a short distance beyond them, and sometimes there are only two or three bays. Sometimes there is a girder or wall at the outer edge and sometimes there is not. Manifestly, when the outer row of columns is reached, there must be—in order to satisfy the theory—something to take the place of the continuous slab that will supply its equivalent. A girder or wall on the line of the columns does not perform this office, as a girder cannot hold a slab in a fixed horizontal position as though it were a cantilever. A continuation of the slab beyond the column line cannot supply the condition necessary, unless it is uniformly loaded with just the proper balancing load. This uniform load, however, would of necessity include some live load, and there is no law compelling an owner to dispose the load on his floor in any given way.

The idea that the slab is a cantilever over the column means one of two things. Either the load on the slab must be properly balanced, or the columns must be subject to a bending moment due to live load to one side of the column line. To assume that the live loads are always properly balanced over the columns is to restrict the owner in the free use of his structure and to demand something unthinkable. To make the columns carry the cantilever load of the slab is to subject them to stresses for which they are never calculated.

On page 67* the author says:

"The stress at 11 in the wall panel, *C*, was larger, and amounted to 20 500 lb., which probably was due to the cantilever action at the wall being less than that exerted by the usual column heads."

Here is a confession that the column heads, which means, of necessity, the columns, are depended on to take cantilever stresses from the floor.

It may be urged that the stiffness of the slab will make it capable of taking at least a part of the bending moment beyond the column line. It cannot possibly take all of it without either infinite stiffness or articulation in the column. Any theory is wrong that does not follow to an ultimate and adequate support or resistance the loads or strains assumed on part of a structure. A slab theory that ignores the effect of the slab on the columns, and leaves the reader to infer that these columns are centrally loaded, is wrong.

Another part of the theory that is wrong is the assumption that Poisson's ratio has any application to the steel reinforcement. The author's theory for the mushroom slab assumes that the steel reinforcement is equivalent to a continuous flat sheet of steel. This is contrary to fact and reason. A set of cross-rods does not in any sense fulfill the conditions that make it equivalent to a single sheet of steel.

* *Proceedings*, Am. Soc. C. E., January, 1914.

Mr. Godfrey

It is freely admitted that when a sheet of steel is pulled in one direction it contracts in the other direction. As to whether this contraction is equivalent to a compressive force that would give the same amount of shortening is another question. If this is applied to two sets of transverse rods, an entirely different condition exists. The only medium between the sets of rods is the concrete. In a previous discussion* the author has stated that for the mushroom type Poisson's ratio is five-tenths. For the type that he delights to honor, then, here is what this statement means. A 1-in. square rod stressed to 16 000 lb. is pulled crosswise by the concrete 8 000 lb. for each inch of its length. The concrete might have an adhesion one one-hundredth of this amount. Where does the other ninety-nine-hundredths come from?

In asking the Profession to accept this mushroom theory, the author is asking us to consider the laws of matter suspended for this particular type, or else that there are special laws that apply to this special combination of materials. The theory of flat slabs is hard for any disinterested engineer to accept; let us look at the tests.

The author finds agreement between his theory and the result of tests. According to his statements, deflections and extensions in the steel would seem to bear out his theoretical deductions. This apparent agreement is explained by two things, one is tension in the concrete, and the other is large columns that are only partly loaded by the tests and, therefore, can lend some of their unused strength toward resisting cantilever action.

There can be no doubt that tension in the concrete plays a large part in resisting bending moments, and consequently carrying the loads. This is true of other forms of reinforced concrete, as well. In a simple beam or slab a very large part of the bending moment is resisted by tension in the concrete; but the accepted theory ignores this tensile strength entirely, and wisely so. This is the reason that deflection calculations for reinforced concrete beams, based on no tension in the concrete, are worthless.

In a flat slab, the tensile strength of the concrete has vastly more influence in supporting loads or resisting bending moments than in a simple beam, because of the fact that the tension is in all directions. Deflection is diminished and steel stresses are reduced by reason of this tensile strength of the concrete, and any theory that tries to explain these results through the supposed influence of Poisson's ratio is false. It cannot be gainsaid that, if a flat slab were cracked along certain lines, the steel stress and the deflection would be found to be very much more than they are in a whole slab, just exactly as the same would be true in any reinforced concrete beam or slab.

Tests on a flat slab, that are manifestly very greatly influenced by the tensile strength of the concrete are falsely interpreted, when the

* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1368.

Mr. Godfrey.

effect of that tensile strength is totally ignored, and, instead, Poisson's ratio (a thing that applies only to "any piece of material which is subjected to stress, and is of such shape that more than one of its dimensions is considerable,"* hence not to a rod) is brought in to explain the low stress in the steel.

If the commercial investigators frankly admitted that the tensile strength of the concrete is their mainstay, users of the flat slab would adopt it with their eyes open, if they had the temerity to use it.

The author has published† a test on a mushroom slab which purported to show the great strength of this style of construction and its agreement with theory. This slab, though only 18 ft. square, was supported on four comparatively enormous and very short reinforced concrete columns. It was loaded with balancing loads, that is, the first and second increments of the load were partly within and partly without the square enclosing the columns, which was 12 ft. each way.

Of course, such columns would have large influence in carrying cantilever loads, because of their 18 in. of width and their small direct load, but this is totally ignored in the author's theory, and all the strength is attributed to the virtue of the system. A building could not be commercially designed with columns about half as thick as their height and a safe load of less than 100 lb. per sq. in. This test is so far from representing anything in practice that it has not even a theoretic interest, especially when the theory absolutely ignores the greatest factors in supporting the loads.

Furthermore, balanced loads on this test slab tell nothing whatever of what the slab would do if the loading were confined within the square enclosing the columns.

The dishing effect of a flat slab is often referred to as explaining the strength exhibited in tests and there is no doubt that for an interior loaded bay, with the surrounding bays idle, the dishing has a large influence. The writer has shown, by tests,‡ that though dishing in an interior panel greatly increases the strength of that panel, a row of loaded panels will not show this dishing, and will not have this large strength. The tests which have been made on flat slabs in buildings have been on one or more interior panels (including in some cases exterior panels supported on walls or girders) but never, so far as the writer can learn, on a complete row of panels across a building. Under such loading, the slab would tend to take a cylindrical shape instead of being dished, and the concrete and the steel will be under very much greater stress.

A flat slab on rows of columns is no better conditioned than the same slab supported on parallel lines of girders. No commercially

* As defined by Dr. Eddy in *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1364.

† *Engineering News*, March 27th, 1913.

‡ *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1358; and *Engineering and Contracting*, July 2d, 1913.

designed flat slab will stand up under this criterion, that is, considering the slab supported on two lines of girders and figuring its bending moment as an ordinary slab subject to the common laws of Nature to which other than "flat slabs" are amenable. The writer has made these statements a number of times; they have never been controverted. This is the case of the flat slab in a nutshell. It stands or falls on the criterion just mentioned. In one notable case it fell with disastrous results. Mr. Godfrey

The most serious aspect of this reliance on tension in concrete is in the case of rolling or jarring loads, as for a freight terminal or a viaduct. Repeated jarring of the load will in time crack the concrete. Then the steel will get its full load, which is several times that for which it is calculated, and trouble may be expected.

H. E. Eckles, M. Am. Soc. C. E. (by letter).—The writer is glad to note the author's attempt to formulate conclusions in a more practical way than has characterized some of his past efforts. The assumption of a value for Poisson's ratio for steel and concrete in combination at four or five times its probable value, as shown by the published results of experiments with these materials in combination, and one-third larger than the sum of the ratios for the materials when not in combination, and used by the author in one of his published works in the derivation of formulas similar to those presented in this paper, accounts for some of the discrepancies between former theories and conclusions based on facts. The variations in flat-slab design and the growing importance of this type of construction are so great that attempts to formulate conclusions should be made with care, and with a view to general applicability rather than in support of a particular type of construction. Mr. Eckles.

Much has been written regarding the action of flat plates under loads, and the writer believes that considerable of what has been put forth as a proper basis of design by advocates of reinforced concrete flat-slab construction is erroneous, and that the formulas proposed by the author in this paper give resultant stresses in the steel much less than would be developed under working conditions and proper test loading.

It should be noted that the results found by using the proposed formulas are compared with the observed stresses in the steel in the slab of Panel D when there was little load on the panels adjacent to it. In the adjacent Panel A the equivalent uniform load was about 260 lb. per sq. ft. of floor; in Panel C it was about 30 lb. per sq. ft.; and in the two other adjacent panels there was no load whatever. Our knowledge of this construction warrants us in believing that the effect of the surrounding floor in supporting the load of a single panel is quite large.

Mr. Eckles. The writer recently conducted a test of a floor of the "mushroom" type of construction, having panels of the same dimensions and with a slab 7 in. thick, reinforced, however, with a larger percentage than usual of slab steel. The design load was 200 lb. and the superimposed test load was 400 lb. per sq. ft. of floor. The results of this test showed a remarkably large effect on the deflection at the middle of the panel, when parts of the adjacent panels were loaded with a full test load along two sides.

The adjoining panels were of the same dimensions and reinforcement as the one loaded. The panel tested was at the corner of the building, supported on two sides by wall beams, and corresponding to that of the corner panel shown in Fig. 1, having Columns 9, 10, 11, and 48 at the four corners.

In the first stage of the test, the panel only was loaded. Care was taken to leave clear passages so as to prevent any arch action.

The second stage consisted in loading areas 4 ft. 6 in. wide adjacent to two sides of the loaded panel, for the full length of the side of the panel, with the same uniform load as that on the panel. This placed a full test load practically over the full width of the belt of bars extending from column to column and parallel to the sides of the panel. After the placing of this additional load, the measured deflection at the center of the panel was exactly twice that found when only the full area of the panel was loaded. The measurements were made very carefully by two direct methods, to avoid any error, and a period of 40 hours was allowed to elapse between the time of the loading of the panel and the loading of the adjacent strips of floor. It seems probable that the difference in the deflections would have been somewhat larger had the width of the adjacent loaded strips been greater.

This is in harmony with the results of tests given in papers presented* some months ago, in which it was shown that loads are distributed laterally for a distance beyond the boundaries of the loaded area, in some cases amounting to more than one-half the span of the slab. Similar conclusions have been reached as a result of other tests. The method of arranging the reinforcement in this type of construction, shown in Fig. 1, with the belts of bars overlapping each other near the columns, would seem to indicate clearly that, for a single loaded panel, where the side belts are only partly loaded, the effective span of the diagonal belts would be decreased or largely modified. This effect would be enhanced still further by the fact that the side belts are of shorter spans and consequently subject to less deflection. This is precisely the effect of the "drop," used in the Larkin Building, and would seem to account for the approximate agreement of the results

* At a meeting of the American Society for Testing Materials.

of the calculations by the author's use of his formula, with those of the Larkin test, where the load extended over several panels. Mr. Eckles.

It has been pretty clearly established, by the numerous tests made, that the points of contraflexure are fairly close to that which the common theories of flexure would indicate, considering an elementary width of slab extending from column to column to act as a beam.

In the "mushroom" system this point is much nearer the column than in cases where the "drop" is used, and calculations made on this basis give results considerably in excess of those by the author's proposed formulas, based on partial test loads. The author has fallen into error through his disregard of the well-established laws of flexure, with the result that the conclusions in his paper are erroneous.

The advantages of the flat-slab type of construction are obvious. With the same quantity of material, the structure can be made approximately of the same strength as when the usual slab-and-beam type of construction is used, without excessive deflection, and with the added advantage, in many cases, of greater clearance, or a reduction of the cost by a less height of structure being required. With these facts in view, it is questionable practice to advocate a flat-slab type of construction designed under specifications inconsistent with what is recognized as good practice in the more common slab-and-beam type.

One of the disadvantages of the "mushroom" system without the "drop" is the lessened strength in shear of the slab around the column head. At the beginning of the paper the author has advocated the use of small percentages of reinforcement, though his reasons for this are not apparent. The quantity of reinforcement often used in the "mushroom" system is one-fourth of 1%, and, considering the possible effect of the overlapping belts, with this percentage the neutral axis of the slab would be approximately three-tenths of the effective depth from the compression face at points near the column. In a slab of 8 in. effective depth, the neutral axis would be about 2½ in. from the face of the slab.

The compression side of the slab is the only part of its cross-section which is effective in shear; the shear developed on this part of the slab around the top of the column is a measure of diagonal tension, and is of very high unit value. Where a "drop" is used, this unit shear is greatly reduced. With the same total quantity of reinforcement and a "drop" of half the thickness of the slab, the unit shear is reduced approximately one-fourth.

Sanford E. Thompson, M. Am. Soc. C. E. (by letter).—Dr. Eddy fails to bring out the following important conclusions from the tests on the Northwestern Glass Company's Building: Mr. Thompson.

1. The reinforcement of the slab at the column head is entirely inadequate for the design load, so that the tensile stresses in

Mr. Thompson.

the steel and the compressive stresses in the concrete are excessive.

2. The gauge lines at the column head were not placed properly, so that the readings of the stresses do not represent the maximum stresses in the steel at the column head.
3. Tests made on single panels, or with the loading similar to those on the Northwestern Glass Company's Building, Loads 5 to 7, do not produce the largest stresses in the most important part of the structure, that is, at the column head.
4. The wall panels should be designed differently from the inside panels.

Stresses at the Column Head.—By the nature of the construction, the flat slab derives its strength from the rigidity of the column head. For this reason, it is absolutely necessary to design the slab so that the tensile and compressive stresses at the column head are within working limits. In analyzing the results from the tests, it is most important, therefore, to pay close attention to the stresses at the column head.

For the column head, the most unfavorable position of the loading is when all the panels around it are fully loaded. In the Northwestern Glass Company's Building, the only loading that caused the most unfavorable condition at the column head was when Load 3 was applied, and then the total load per panel was from 93 000 to 97 000 lb., though the panels were designed for a total live load of 102 400 lb. Even for this smaller load, however, the stresses in the steel due to the live load alone reached 22 000 lb. per sq. in. If to this is added the stress due to the dead load, the total stress for the design load is more than 50% greater than that usually allowed in reinforced concrete construction. The stress referred to is at Point 111 (Table 12). Stresses just as high undoubtedly would have been found elsewhere, if the other gauge lines had been properly placed over the edge of the head, as explained later. That this stress was not abnormal is indicated by the uniformity with which it increased under the load and then decreased when the load was removed.

Dr. Eddy attributes the high stress at Point 111 to the fact that:

"It was necessary to pry the ends of the rods upward forcibly and hold them in this position by blocks under their extremities, thus putting them under considerable initial bending stress."

As the gauge line, 111, was on a straight bar belonging to the diagonal band, it is difficult to see how lifting the steel in the bands, which were not attached to anything, puts any initial bending stresses on it. Even if initial stresses existed, they would not have affected the readings of the stresses, as extensometer measurements do not indicate the actual stresses in bars, but only the differ-

ence between the stresses at the initial reading and at the respective reading under load. Any initial stress, like stresses due to the dead load, is not included in this difference. Mr. Thompson.

Mr. Eddy states, further:

"At 111, Column 36, on the edge of the loaded area, the abnormal value of 22 400 lb. was reached under Load 3. Load 5 shows no such large increase over Load 3 of observed stress in the slab rods of the column heads in Panel D as was to be expected by practically doubling the loading. This fact is apparently inexplicable."

This statement is absolutely misleading. Point 111 is in Panel C, and this panel received its largest loading with Load 3. With Load 5, Panel C had a load of only 39 lb. per sq. ft., as may be seen by referring to the detail of loading in Fig. 6. It is entirely obvious, therefore, that the stress at this point ought to be less with Load 5 than with Load 3.

Faulty Location of Gauge Lines.—The location of the gauge lines over the column head is unfortunate. As is evident from Fig. 3, all gauge lines, except 103, 107, and 111, are inside the column head. The readings, therefore, on these gauge lines cannot represent in any way the actual maximum stresses in the reinforcement of the column head.

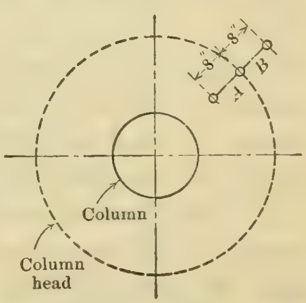

Fig. 20.

The writer has in his possession tests made by one of his associates on a similar construction, where readings were taken on gauge lines placed as shown by Fig. 20. The average results from this test gave at *A* a stress of 11 000 lb. per sq. in., and, at the same time, the stress at *B* was 24 000 lb. per sq. in. Similar results were obtained in the Worcester test, made under the direction of Mr. Brown. It is evident, therefore, that for gauge lines located like those used in the Northwestern Glass Company's Building, the results are much smaller than the maximum stresses.

This rapid decrease in stresses explains why, in Table 12, all those except 107, 108, and 111, were very low. The low stress in 103, which was placed outside the column head, is evidently caused by the fact that the gauge line was too far from the column head, and the bar was one of the outside ones of the diagonal band.

That Dr. Eddy realizes that the stresses in the rods inside the cap are much smaller than those in the same rods just after they leave the cap, is evident from the following:

"Although that part of the length of the rod which is inside the cap has its elongation prevented by the mass of the cap, the part outside must have its elongation correspondingly increased to compensate for this loss, and, on the whole, be equal to that of the rods beside it."

Mr. Thompson.

In analyzing the results from the test, however, he accepts the stresses inside the cap as the maximum stresses.

Stresses in the Concrete at the Column Head.—Dr. Eddy does not mention the stresses in the concrete at the column head. In the first part of his paper, he analyzes the relation between the stresses in the steel and in the concrete, and the influence of the percentage of steel on the stresses in the concrete. This analysis would lead one to believe that the stresses in the concrete, in the tests described later, are within working limits. This, however, is not the case. The case of "under reinforcement," mentioned by Dr. Eddy, exists within the panel. At the column head, on the other hand, the percentage of steel in the bands alone is four times that of the steel in each band. Whereever the bars overlap, the percentage is still larger. In designs similar to those discussed in the paper, the percentage of steel at the column head is between 1.5 and 2. For these percentages of steel, the stress in the concrete, corresponding to the working stress in the steel, must exceed the allowable working stress. This is evident from Table 1.

It is of great importance for the safety of the structure to keep the compressive stresses at the column head, as well as the tensile stresses in the steel, within working limits. Therefore, if a large percentage of steel is used at the column head, as is always the case in this type of construction, compression steel is indispensable.

In the Northwestern Glass Company's Building, an idea of the stresses in the concrete due to live load alone may be had if one calculates the stress in the concrete corresponding to about 1.3% of the steel stressed to 22 000 lb. With the dead load included, the stresses in the steel and the concrete would be at least 25% greater.

Loads 5 and 7 do not Produce the Worst Condition at the Column Head.—It is unfortunate from a scientific standpoint that the loading of all the panels was discontinued as soon as Point 111 received high stress. By further loading of all the panels, their true strength would have been obtained, but with the loadings as used, the results are simply misleading.

Loads 4*a* and 5, for instance, show Panels B and D loaded and Panels A and C unloaded. With this load, the stresses at the column, instead of being taken by the steel in one-fourth of the circumference of the column head, that is, by the steel tributary to that panel, were taken by the steel in one-half of the circumference. The correctness of this statement may be seen by reference to Table 12 where it is shown that although Panels A and C were not loaded, Points 103, 107, and 111, showed considerable stress. From this it is evident that the loading of Panels D and B was carried at the column head, not only by the steel belonging to Panels D and B, but also by the steel in Panels A and C. The stresses at the column head, therefore, with

this kind of loading, are much smaller than would have been the case had all spans carried loads of equal intensity. Mr. Thompson.

Other Tests on Mushroom Floors.—The student of this type of construction is struck by the paucity of reliable test data on the particular type of construction discussed by Dr. Eddy. Many tests are constantly referred to in printed literature, but on closer examination one is surprised to find that they bring out the stresses everywhere except in the most vulnerable part of the construction.

The test to destruction carried on by C. A. P. Turner, M. Am. Soc. C. E., or on his behalf, described in the engineering papers and more fully in Dr. Eddy's book, might have elucidated many mooted questions. The idea of a test panel with projections, if loaded properly, was a very good one, because the load on the projections could have been arranged so that it would have an effect almost similar to loads on the adjoining panels. The idea, however, was not carried out, and the slab was broken by loads placed in the center panel with comparatively little load on the projections. The question of the stresses in the most important part, that is, at the column head, therefore, was neglected, with the consequence that the results of the test are simply misleading to any one not well versed in the subject.

The test of the St. Paul Bread Company's Building is another instance in which everything was tested except that which is ordinarily the weakest part of the construction.

Formulas.—Dr. Eddy's formulas do not agree with the results of the tests in the most important place, *viz.,* at the column head. Although he calls them "theoretical formulas", the theoretical foundation is nullified by the many assumptions which do not agree with the actual conditions, so that the final results can lay no claim to being theoretically correct.

One of the assumptions for which no clear reason is given is that for the value of Poisson's ratio, K. As explained in Dr. Eddy's book, that ratio, for concrete alone, varies from 0.1 to 0.2. For steel alone this ratio is about 0.3. For a flat-slab construction, however, Dr. Eddy considers that neither of these two values is large enough, and he accepts a value of K equal to 0.5. To support this value, he has given a beautiful formula, but does not explain how two materials acting together can change their nature entirely, and how the top portion of a concrete slab, by merely being provided with steel at the bottom, can take on a lateral expansion equal to one-half the compressive deformation under stress. The writer does not see how this statement can be accepted by any one acquainted with the nature of the material, and yet on this assumption hangs the most vital element in the results.

The stresses for the steel in the diagonal and rectangular bands are calculated from formulas derived without regard to the size of the

Mr. Thompson. column head, and, according to Dr. Eddy, would apply to a construction with a column head equal to a sharp point and one of any size whatever. This assumption, on the face of it, is erroneous. The stresses in concrete at the column head are simply neglected.

In determining the stresses in steel over the column head, Dr. Eddy states: "The stresses in the middle rods of each belt, consequently, are increased abnormally for this reason just as it leaves the cap."

This statement, however, does not prevent him from continuing:

"Instead of attempting to determine this increase by some intricate investigation, it will be simply assumed that the stress at this point in the middle rod does not exceed that in the outside rod of the belt at a point opposite the center of the cap."

The available tests prove the correctness of the first statement. They show, however, that the stresses in the outer bars are much smaller than those in the middle bars. Dr. Eddy's assumption, therefore, is without foundation and, as a consequence, the results from the formulas do not agree with the tests. In the Northwestern Glass Company's Building, by using the actual live load on the panel, there is obtained by his formula a stress, $f_s = 15\,400$ lb. per sq. in., while the actual stress due to live load is at least 22 000 lb. per. sq. in.

In discussing the tests, Dr. Eddy uses, in his formulas for the total load in the panel, a much larger load than was actually on the panel. As a justification for this, he states that portions of the panels near the column heads were not loaded. He overlooks the fact, however, that there were two center aisles about 18 in. wide running across the center of the panel and thus taking away the load from the place where it would be most effective in producing bending moment. The use of the larger load is still more unjustifiable in calculating the stresses over the column head, because the stresses there are more affected by the amount of load in the panel than by their position. It appears that the same load uniformly distributed would not have caused larger stresses than was caused by the load placed as shown in Fig. 5.

In discussing the Larkin Building, Dr. Eddy makes the following statement:

"It would require a load of more than 1 200 lb. per sq. ft. on such a mushroom slab to produce a maximum unit stress on the slab rods as great as was caused in the Larkin slab by the test load of 738 lb. per sq. ft."

This, of course, is calculated according to his formula, which, as has been shown, does not agree within 45% with the actual results. This claim is only another expression of Dr. Eddy's faith in the miraculous properties of this particular type of reinforcement; but, to engineers familiar with the design of reinforced concrete, it seems at least open to discussion.

As a conclusion, it appears, from a careful study of the tests, that a design similar to that of the Northwestern Glass Company's Building does not have the proper factor of safety required in reinforced concrete construction; that the stresses in the steel, as well as in the concrete, as is evident from Table 12, exceed those used in conservative designs; and that the formulas given by Dr. Eddy do not appear to agree with the results with the accuracy claimed by him; in fact, as stated before, the difference between the results from the formulas and the actual stresses reaches the large amount of at least 45 per cent. Mr. Thompson.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

GROUTED CUT-OFF FOR THE ESTACADA DAM.
Discussion.*

By Messrs. S. Howard Rippey, S. C. Hulse, Frank R. Fisher, H. L. Coburn, and William H. Cushman.

S. Howard Rippey, M. Am. Soc. C. E. (by letter).—The deep interest in the subject of grouting dam foundations, as shown by numerous private inquiries from engineers in America and abroad concerning the Clackamas River experiments, renders it certain that this paper will be generally appreciated by the Profession, and it is to be hoped will develop a broad discussion of the general subject of dam foundation treatment. Although this subject is always of interest to those engaged in hydraulic work, its seriousness is from time to time given added significance by such unfortunate occurrences as the failures of the Austin and Stony River Dams, as well as of other less conspicuous structures, showing that perfectly well designed and constructed concrete dams of any type may be wrecked as a result of inadequate preparation of their foundations. Mr. Rippey.

The detailed information given by Mr. Rands concerning the practical execution of the grouting programme at Estacada should be of value to others facing similar problems, but the writer fears that some of the conclusions reached by the author and the personal opinions expressed, may tend to indicate to others undue difficulties and limitations in the general applicability of the grouting method to dam problems, and to that extent discourage the use of a method which in many instances may represent the only economically practicable solution of a development project.

In order, therefore, that the results obtained at Estacada, and the conclusions drawn therefrom by Mr. Rands, may be properly appraised

*This discussion (of the paper by Harold A. Rands, Assoc. M. Am. Soc. C. E., published in January, 1914, *Proceedings*, and presented at the meeting of February 18th, 1914), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

Mr. Rippey. by others interested in the treatment of deficient foundations, it is proper to outline, as briefly as possible, sufficient of the history of that project to indicate the unfavorable character of the governing conditions under which the work was executed.

As indicated by Mr. Rands, the use of grout at Estacada was prescribed by the writer, following thorough study of the foundation requirements at the site of a proposed large reservoir and power dam on Clackamas River, some 5 or 6 miles above Estacada (known as the Upper "Clackamas" or "Second Clackamas" project). At this site a mass concrete dam about 150 ft. high was found to be desirable as part of the most efficient plan of power development and flow equalization, but the character of the foundations (similar to those found later at Estacada), was such that the writer felt that the construction of such a dam on them would not be in accordance with any known precedent, good judgment, or the dictates of good practice, unless the objections could be overcome. It thus became necessary to decide whether to modify the development scheme so as to avoid the necessity for a high dam (as, for instance, by a low dam and a long flume to create the desired head), or undertake to improve the character of the foundations so as to justify the execution of the more efficient and satisfactory plan of development.

At this juncture the writer decided that the use of cement grout, injected under pressure, offered a possible solution of the problem of foundation treatment, and probably represented the only practicable method of treating this remarkably heterogeneous formation, so as to justify the construction of a high masonry dam on it. Test-pit and core-drill investigations in progress indicated the absolute need for novel measures in order to render the foundations fit for the proposed construction, and the formations encountered were such as to make the writer believe that the grouting method could be applied successfully.

In order to acquire a more comprehensive idea of the possible range of geological conditions to be encountered than was feasible through the local borings in progress and by superficial examination of the canyon in this locality, the writer, fortunately, was able at this time to secure a very thorough geological report from Professor J. S. Diller, of the U. S. Geological Survey, who made a personal examination and study of the canyon for the purpose. In view of the questions raised by Mr. Rands, the following abstract of Professor Diller's report is considered pertinent to the discussion:

"The rocks of the canyon walls are of four forms, volcanic breccias, lava sheets, volcanic dikes and terrace gravels. * * * The volcanic breccia (bed-rock) is made up of unassorted angular fragments of lava, andesite and basalt of various colors, ranging in size from dust particles and grains of sand to large rock fragments many feet in diameter.

* * * Sheets of solid non-fragmental lava forming part of the bed-rock and outcropping on the slopes of the canyon occur within and between the great sheets of volcanic breccia. Some of the lava sheets are basalt * * * generally very porous. * * * Nearly vertical dikes of basalt cut up through the sheets of volcanic breccia and lava, and their outcrops on the surface have the direction of N. 65° W. approximately parallel to the general course of the canyon. * * * There is a set of parallel joints, the open cracks of which cut up through the volcanic breccias and sheets of lava about vertically in a direction approximately parallel to the course of the canyon. * * * Such joints may be of considerable extent and form important openings for the circulation of water. Well developed joint cracks of this system were not seen in the exposed bed-rock of the dam site, but they may be expected, and should be carefully looked for where the bed-rock is covered with soil or gravel. It is especially significant that the dikes are approximately parallel to these joint cracks and suggest that these joint cracks may extend to great depths. * * * From the nature of the volcanic breccia, which forms by far the greater part of the canyon walls, it is evident that the drill cores will differ from one another very much when compared. Where the drill goes through a sheet of lava or a large solid fragment it will yield a good core, but where it penetrates the finer material (the volcanic ashes, in which the fragments of all sizes are imbedded), the core fails, the material is pulverized by the drill and washed away, and yet the extent of this material that is washed away is of the greatest importance, for it is the weakest element in the structure and the one which when saturated with water under pressure is most likely to become Engineers' 'soapstone'. Soapstone, properly so-called, does not occur in that region at all, but decomposed lava, volcanic ashes, and clay, all of which when saturated with water may become slippery and would be called 'soapstone' by Engineers, occur locally in the volcanic breccia. Large caverns and cavities, or pockets of loose earth and stones, are not to be expected in the volcanic breccia, but, owing to the manner of accumulation, there may be small openings, and the porosity of the rock is high. It is pervious to water, and for this reason similar material is used for making water coolers. The crushing strength of the volcanic breccia is, of course, small as compared with granite, limestone, and most other rocks, and this, taken in connection with its porosity and the possible existence of undiscovered joint cracks, seems to make a large reinforcement with concrete necessary, in order to furnish strength and prevent seepage as well as erosion. Mr. Rippey.

"The conditions that confront the engineer along the Clackamas River in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade Range from the Columbia River in Oregon to the Feather River in California—one of the most important water-power belts in the United States—and the successful solution of the problem it presents at one point will greatly facilitate the work elsewhere."

At this time only one of the core-drill holes had been subjected to water pressure as part of the original programme of studying the porosity of the foundations. In a report to his clients, covering in-

Mr. Rippey. structions to the Field Engineer, Shirley C. Hulse, Assoc. M. Am. Soc. C. E., the writer said at this time:

"The result of this test shows that 25 gal. per min., at a pressure of 175 lb., have been pumped into this hole for several consecutive hours, and, while coloring matter was used, no points of escape were noted and the behavior of the water surface in the surrounding holes has been so small and erratic as to be negligible. It has been evident since the first holes were drilled that we are dealing with a very porous and permeable foundation condition which will require special treatment to render it suitable for the construction proposed, and the purpose of the pressure tests outlined was to indicate the extent of permeability and the general lines of leakage, so that we might best plan to make the foundation of the dam impermeable. * * *

"In the Middle West, where a number of masonry structures are built on sand or earth foundations, it is customary to cover the river bottom for some distance up stream with a layer of clay and depend on the silt in the river closing up all the fissures and paths of seepage, but it would be very unwise to depend on this method in such a construction as we have under consideration. * * * It is necessary to solidify and render impermeable the formation immediately beneath our proposed dam before we are justified in building a dam at this site.

"It is evident that impermeable foundations are desirable: (a) to minimize the possibility of upward pressure under the base of the dam superstructure (this being a solid concrete dam); (b) to prevent percolation under the dam which might lead to sufficient erosion to involve undermining the structure; (c) to overcome any structural weakness due to the original geological formation, and properly provide a support for the superimposed load; and (d) to avoid waste of water (the chief power asset of the Company) from the reservoir, through, under, or around the dam, instead of through the turbines where it creates K. W. H. available for sale.

"It is * * * evident that we cannot afford to build the plant and find out afterward whether we can make the foundations impermeable, but that we should adopt a programme whereby we may hope absolutely to demonstrate our ability to render the foundations impermeable before the superstructures are built. * * * The general idea involves drilling a double line of holes under the heel of the dam and forcing into each hole grout of such consistency as to percolate through the permeable substructure and solidify it absolutely throughout the entire length of the superstructure, thus making the foundations absolutely solid and also providing the equivalent of an absolute cut-off wall. * * * Cement grout should be pumped into these holes and after time has been allowed for hardening, a third line of holes should be drilled midway between the first two lines and tested under water pressure. If water pressure is applied at or slightly above the hydrostatic pressure to which this rock would be subjected by the water in the reservoir after the completion of the dam, and it is found impossible to force any amount of water through the holes, we may feel reasonably certain that the cement grout has proven effective in making the entire foundation impermeable. * * * The test * * * may show a very large quantity of leakage, and this will simply mean that more holes would

have to be drilled and pumped full of grout. * * * The programme to be followed will be necessarily indicated to a considerable extent by the experience obtained as the work progresses. * * * In order to avoid the uncertainties consequent upon an arbitrary determination of the spacing * * * of these grout holes, as well as of their depth, it appears expedient to select a convenient site where the cores indicate as difficult a formation to make tight as any on the job and to use this site as an experimental laboratory. * * * It seems probable, however, that this experiment may be made as part of the preparation of dam foundations, and the total cost of the work thus kept at the minimum. * * * We still regard it as essential that our ability to make the foundations tight be absolutely demonstrated before the work is started on the dam, and the experimental programme outlined provides for such demonstration." Mr. Rippey.

This report included detailed descriptions and sketches unnecessary to reproduce here.

At about this time the company acquired from other interests the "River Mill" property, below Estacada, and for business reasons decided to push a development there to early completion. The programme adopted by the owners of the properties provided that the detailed surveys, foundation tests, and preparation of detailed plans be made by the local organization and submitted to the writer, as Consulting Engineer, for his criticism and approval. The instructions issued specified: "This particularly applies to the foundation, as no chance or risk must be taken of locating a dam upon unsuitable foundations".

The general character of development adopted was similar to that already planned for the higher-head, upper project, except that the hollow reinforced concrete type of dam, which the writer had selected for two earlier successful developments in New England, was adopted on his recommendation, and the grouted preparation of foundations on the general plan in progress at the upper site was prescribed.

The programme adopted in the interest of saving time, with the writer's hearty approval, was soon violated in many respects by the local organization (the personnel of which has since changed). The construction programme being followed was found to be such as the writer could not approve, and he, therefore, advised his clients that the modifications locally made in the foundation treatment were "beyond the limits of the precautions which should be taken in such an undertaking, and while no one can claim with any assurance that failure will result, I cannot conscientiously approve the programme adopted, nor assume any responsibility in connection with it."

The writer also stated, in another communication:

"The successful treatment of the foundations is dependent to a large extent upon the detailed methods adopted, but it is also dependent upon the spirit with which this work is carried out. Your organization has already expressed the opinion that detailed studies and

Mr. Rippey. grouting are unnecessary, and, further, that detailed programmes prepared in advance of actual exposure of foundations are useless.

"We therefore feel that, aside from the engineering questions involved, your organization has not a proper appreciation of the subject; and further, any programme that may be insisted upon which is not in accord with their ideas would not be carried out in the proper spirit."

The foundation work had been held up for several months because of the writer's inability to approve what was proposed and the unwillingness of the local organization to follow the programme ordered, when the President of the Company suggested that John F. Stevens, M. Am. Soc. C. E., late Chief Engineer of the Panama Canal, be asked to pass upon the questions at issue. As a result of the opinions expressed by Mr. Stevens* (who was practically an arbitrator selected by the local parties), the owners of the property were confirmed in their support of the judgment of the writer, and insisted on the work being properly executed. So much time had been wasted, however, and the urgent need for power was so great, that it became necessary to carry along the grouting work in connection with actual construction of the superstructures in the best possible manner under the circumstances, and the conditions were not ideal for the development of the full possibilities of the use of grout.

The reversal of the local authorities and their reluctant acceptance of the ordered programme produced an attitude which resulted in unnecessary difficulties, obstructions, and personal controversies on the ground, finally leading to another change of Resident Engineers, for

*By the time the situation was formulated for presentation to Mr. Stevens, the local organization had developed a programme for the drilling, testing, and grouting of foundations, coincidently with or subsequent to the construction of the dam and other superstructures, instead of prior thereto, and his formal report, therefore, related largely to a comparison of the methods of execution, rather than to the accepted fundamental necessity for grouting the foundations. The following extracts from his report, however, indicate clearly his attitude on the larger question and his appreciation of the importance of foundation treatment:

"The limited number of test pits and borings which have been made shows clearly that, as far as concerns the area they covered, no well-defined, extremely hard masses of rock exist—that the rock is soft and shades away into clay, the latter appearing under superficial examination much like a half-burned brick. The whole formation, to the non-geologically scientific man, seems to be a mass of soft conglomerate, * * *

"The writer regards it as unfortunate that more test holes were not put down. * * * Enough testing, however, has been done to establish the fact that seams, or fissures, or cavities, do exist where certain of the holes were put down, and therefore it is only fair to assume that they exist where no testing was done; in fact, the only safe assumption is that they exist all over and through the formation, but to what sizes and shapes no one can say. There is no doubt in the mind of the writer that all such seams or cavities can be filled and sealed to exclude water by the proper introduction of cement grout. * * * If liquid cement under 200 lb. pressure cannot be forced into the material, the water under 35 lb. cannot find its way, and the writer concurs in such opinion. * * * Grout to be forced in by compressed air under 200 lb. pressure, the flow to be continuous from start to finish. This the writer believes to be the most practical and best method. * * * the writer believes that the material is, over the greater part of the area, porous enough so that under the heavy pressure the grout would form a practically continuous body, or curtain, from 12 to 15 ft. in thickness. * * * No living man can guarantee the absolute success of the grouting when it is placed."

purely personal reasons. Under all the circumstances, it reflects great credit on the ability and adaptability of all the engineers engaged on the work that they were able to attain as good results as those described by Mr. Rands in the face of aggressive lack of sympathy displayed by the local organization toward the foundation programme. Mr. Rippey.

As his clients loyally supported the writer's judgment in these matters, he regrets the necessity for discussing them in this connection, and at this late day. The Profession, however, is entitled to a knowledge of the actual conditions of the execution of the work, in order that other engineers may not be misguided by the results and by the opinions expressed. Moreover, as the writer's connection with the undertaking has been thus publicly recorded by Mr. Rands, the foregoing explanations properly become part of the record.

It must be understood that although the writer in 1910 declined to be considered responsible for results, in view of the conditions developed through the continued disobedience of governing instructions, he has no reason to question the safety of the final construction, which was nominally completed along the ordered lines, although under many handicaps.

While the Estacada work was in progress, the company acquired other partly developed power properties, and the Upper Clackamas development was therefore postponed before any conclusive results had been reached in the grouting experiments at that site.

About 2 years ago, in discussing the Estacada Dam in relation to the original experiment, the writer stated:

"It will be evident that the question of uplift pressure under a dam of this type is practically negligible, and, as the static head created is only about 83 ft., the other risks incident to a permeable foundation become proportionately smaller. As an indication of the porosity of the foundation material, it was found that the average leakage through 43 holes each 50 ft. deep (individually tested) was at the rate of 82½ gal. per min. under an average pressure of only 17.7 lb. per sq. in. The problem here becomes that of reducing the permeability to reasonable limits to overcome any structural weaknesses, prevent objectionable erosion and limit waste of water, without insisting upon the degree of tightness which would be necessary if the upward pressure under the dam base were a factor. The work was very urgent and the construction programme was modified in numerous respects to meet the exigencies which developed, but the dam was completed in November, 1911, and the foundation treatment appears to have met the practical requirements, although it was not pursued to the extent required to entirely eliminate upward pressure (had a solid dam been adopted).

"The holes were not uniformly spaced throughout the foundation, being located as experimental tests and the related construction work indicated best under the circumstances, and while the dam has been successfully completed and the plant is in regular operation, it is probable that the foundation treatment could be more thoroughly and

Mr. Rippey. economically effected in another case if taken up vigorously during the preliminary stages of a development so as to be independent of construction complications. Moreover, the practical experience obtained should greatly facilitate adapting this method to another development. It will be quite obvious, however, that each site will have its own peculiarities, and radically different treatment may be required."

Although the writer thus recognized the incompleteness of the grouting results attained as a result of the conditions described, and the degree of subordination of the grout work to the construction programme, which existed, it does not appear that the results were as negative as might be inferred from some of the personal conclusions reached by Mr. Rands. The very large quantity of cement introduced into the foundations necessarily represents an improvement in the formation to that extent, aside from any interpretation of pressure-test results, and, as Mr. Rands states, tests made after completion disclosed no leakage of water past the cut-off as the result of a hydrostatic head of from 80 to 90 ft. above the dam.

Although the final official report of Mr. Frank R. Fisher, who was Resident Engineer in charge during the period of completion of the development, indicates clearly that the grout did not produce impermeability over certain limited areas, it shows very definitely the practical sufficiency, for the purpose, of the results attained, as indicated by the following quotations therefrom:

"The excavation of the trench for the supplementary cut-off wall, in the locality where grouting had previously been done, afforded an excellent opportunity for observing its effects, as many seams were exposed, and all of them proved to be well caulked with cement. None of any magnitude were observed, the largest ones averaging about $\frac{1}{2}$ in., and varying from that down to $\frac{1}{16}$ in., the smaller ones also having taken the grout freely.

"There was a total of 29 proving holes put down on this [the right channel] section, and, discarding the tests on two of them that represented excessive leakage that was only local, the average for the remaining 27 was 3.6 gal. per hole. In material of this kind this was not considered excessive, and it is reasonable to assume that for all practical purposes the cut-off is effective on the right channel section.

"*Conclusions.*—The final conclusions to be drawn * * * may be stated as follows:

"*1.*—It is not feasible to accomplish an absolutely impervious cut-off by this method, in foundation material of the character that exists in this region. The method apparently is effective in closing up seams and crevices of appreciable size, as all such exposed by the excavations made subsequent to the grouting were tightly caulked with cement.

"Where seepage occurs through a more or less porous formation, or through veins of compact broken rock mixed with sand or a certain kind of clay rock, very much seamed, but with the joint or cleavage faces in close contact, all of which have been observed to exist over very

limited areas in this locality, the grout has but slight effect in benefiting the condition. Material of this character acts as a filter, the cement remaining in the hole, and it cannot be diffused throughout the surrounding mass. Mr. Rippey.

"*2.*—While the grouting did not result in an impervious cut-off, the amount of possible seepage through the foundations was very much reduced thereby and it may be considered tight to the extent that erosion and appreciable waste of water would not occur.

"*3.*—The tests on the two holes put down inside of the dam, after the filling of the pond, indicate the presence of some upward pressure in the foundations. It is questionable, however, whether this could be prevented, even with an impervious cut-off 50 ft. in depth, for the indications are that the material in this particular locality is such that water would travel downward and pass under any depth of cut-off that it would be practicable to accomplish.

"*4.*—As it has been proven that the foundations cannot be grouted absolutely tight, it is obvious that but little good is accomplished in attempting to reduce still further what might be termed a reasonable rate of seepage by the introduction of additional holes."

The interest manifested in the Clackamas grouting experiments and the successful adoption of similar methods elsewhere clearly demonstrate the fundamental merit of this method of treating deficient foundations. Indeed, in some cases it appears to the writer to afford the only practicable means of rendering certain formations satisfactory for the construction of high solid masonry or concrete dams. The problem of eliminating upward pressure under the base of solid dams, the significance of which was emphasized in the discussion of a paper* by the late C. L. Harrison, M. Am. Soc. C. E., is peculiarly susceptible of solution by the grouting method in certain formations. In the course of the discussion on that paper, Arthur P. Davis, M. Am. Soc. C. E., said:

"Recent experience has shown the feasibility and efficacy, in some cases, of closing the crevices in the foundations wholly or partly by grouting them under pressure. This was accomplished successfully at moderate cost on the Ashokan Dam, and on several others of recent construction. The most striking instance of this kind which has come to the writer's attention is the Clackamas Dam, in Oregon, which was built on a foundation of semi-indurated volcanic ash, which was checkered in all directions by innumerable fissures, and, furthermore, was so soft that percolation was likely to cause destructive erosion. A triple line of holes was grouted along the up-stream toe of this dam, and recent information is that, since the dam has been in use, no perceptible percolation has taken place.

"The effect of such grouting is not easy to foretell, and, like all other underground conditions, must be estimated with extreme caution."

Incidentally, it was the failure of the local organization to recognize the necessity for this extreme caution, as emphasized by so ex-

* "Provisions for Uplift and Ice Pressure in Designing Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 142.

Mr. Rippey.

perienced an authority as the Chief Engineer of the U. S. Reclamation Service and insisted on by the writer, in connection with the Clackamas work, which led to the unfortunate lack of sympathetic co-operation at Estacada.

A private letter from certain foreign engineers who had inquired about the Clackamas grouting experiments, from which the writer does not feel at liberty to quote directly without permission, reported the complete success of grouting to create an impermeable mass of a defective formation through which water freely escaped from a reservoir. It appears that the greatest success was secured by using a very dilute grout mixture, starting with quite low pressure, and finishing with pumped injection, whereby a certain amount of shock was applied which tended to overcome any temporary obstruction. It was demonstrated that the grout, which set well, traveled more than 100 ft., in plan, and the results appeared satisfactory in the highest degree.

In a recent article in a technical journal,* D. W. Cole, M. Am. Soc. C. E., Project Engineer, U. S. Reclamation Service, describes the application of the grouting method to the foundations of the Lahontan Dam of the Truckee-Carson Project. The writer understands that Mr. Rands was in direct charge of the grouting work (following the completion of the Estacada work), and Mr. Cole describes the results as "excellent", evidently considering, in advance of the completion and filling of the reservoir, that they were successful in accomplishing their purpose. He refers to the fact that it was proven that the grout hardened in place and gives evidence of the effectual sealing of fissures and seams and the solidification of the foundations adjoining the grouted holes.

Another article, in the same journal,† describes the grouting work done on the foundations of the Olive Bridge Dam of the Ashokan Reservoir, part of the Catskill water supply project for New York City. Two definite planes of slight seepage, 40 and 60 ft., respectively, below the creek bed, were found. It appears that 1 439 cu. ft. of thick grout were injected (under comparatively low pressure), of which it was estimated that 1 100 cu. ft. entered the seams. No proving tests seem to have been made.

Ample evidence exists to demonstrate the broad possibilities of the grouting method in its application to the solidification of defective foundations, and it remains for the engineer in each case to adapt it to local requirements. Of course, no general engineering method or formula can eliminate the need for experienced judgment in its detailed application, and, in work of this character, sympathetic co-operation is necessary, down to the humblest participant in its execution.

* *Engineering Record*, March 29th, 1913.

† *Engineering Record*, April 8th, 1911.

Mr. Rippey.

The writer believes that the grouting method affords many economical possibilities in water conservation and power development. It may permit the creation of an operating head entirely by a high dam, instead of by a low dam and a long, and perhaps leaky, flume; it may permit the selection of a site for a dam where the topography is favorable but where the natural foundation is impossible or not as satisfactory as at some poorer power location on the stream; and in many ways may be developed to render reservoir and power problems more flexible.

Even where the security of the construction may not be in question, the prevention of waste by percolation may amply justify the grout treatment to secure impermeability. At the Upper Clackamas, for instance, it was estimated that a saving of only 26 sec-ft. would justify an expenditure of $80 000, aside from any of the collateral advantages (there real necessities) of preventing the flow of water under the dam.

It is difficult to imagine any alternative method of foundation treatment in which the range of possibilities is as great as in grouting.

It will be observed that an essential feature of the grouting programme proposed by the writer is the ability to demonstrate by test, before building the superstructure, that the solidification of the porous material to a satisfactory degree of impermeability has been effected. The holes drilled by rotary drills for securing core specimens, and thus studying the foundations, were utilized later for the hydrostatic pressure testing, for washing out loose and soluble material, and for the introduction of grout under pressure. In the peculiar and exceedingly variable material encountered, it was found that in some cases communication existed between holes as far apart as 70 ft., and in other cases grout could not be made to permeate the material sufficiently unless the holes were very close together. The result was that a greater number of grout holes were required than was tentatively assumed without any experimental knowledge whatever, and a greater number than were required for core studies alone. This fact suggests that, in other applications of this method to large undertakings, only sufficient core holes be drilled by rotary drills to permit proper studies of the geological formation and general foundation structure, and that the remaining holes be sunk purely for grouting and testing purposes by apparatus which will perform the work more rapidly and economically.

The nature of the operation involved in securing cores is such as to necessitate slow progress and relatively high cost per foot of drilling, and when the writer reached a point on the original work where he was not specially interested in securing additional core samples, and where the results showed the need for closer spacing of grout

Mr. Rippey. holes than was thought from the general appearance of the rock and its permeability to clear water would be necessary, he considered the use of drills which would perform the remainder of the work more expeditiously and at less cost than was possible with rotary drills made primarily for securing cores.

The average rate of progress with the core drills was about 13 ft. per 10-hour shift, and it was evident that if the further work was confined to plain drilling it could be done much more rapidly. It appeared that the future drilling might best be done with percussion drills with guided rods arranged so as to drill holes 6 in. in diameter and about 50 ft. deep, approximately plumb, at a high rate of speed. A number of drills were found on the market which could be mounted to accomplish this work, but there was great difference of opinion among the manufacturers as to the possible rate of drilling in this material, estimates running from 25 to 150 ft. per 10-hour shift. At this juncture the development was postponed, for the business reasons stated, and the subject was not pursued further, but there is evidently opportunity for great saving along these lines.

The costs of drilling and grouting at the Estacada Dam, as given by Mr. Fisher in his final report, and by Mr. Rands, on a somewhat different basis, in his paper, and the cost of grouting given by Mr. Cole for the Lahontan Dam, all reduced to a unit basis, are given in Table 5.

TABLE 5.—Cost of Drilling and Grouting at Estacada and Lahontan Dams, per Linear Foot of Complicated Work.

Labor and materials.	Estacada Dam.		Lahontan Dam.
	Fisher.	Rands.	Cole.
Labor, drilling	$0.58	$0.59	$0.93
Labor, grouting	0.18	0.18	0.29
Cement	0.12	0.12	0.31
Repairs and supplies	0.17	0.17	0.23
Plant	0.30	....	....
Plant depreciation	....	0.15	0.35
Power	0.05	....	0.03
Other items	....	....	0.94
	$1.40	$1.21	....
Salvage on plant, Credit	0.17	....	....
Direct cost	$1.23	$1.21	....
Total field cost	....	....	$3.08
General plant, etc	0.32	0.45	0.12
Coffers and pumping	....	0.15	....
Engineering and superintendence	....	0.19	0.27
Clerical and office	....	....	0.10
Total cost per foot	$1.55	$2.00	$3.57

It is of interest to note that the cost of cement per foot of hole at Lahontan was more than $2\frac{1}{2}$ times that at Estacada; examination of the data, however, indicates that cement cost about $2.77 per bbl. at Lahontan and only $2.20 at Estacada. The actual quantity of cement used per foot of drilled hole appears to have been as follows: Mr. Rippey.

Estacada.—34 038 ft. of holes: 1 942 bbl. = 0.057 bbl. per ft.

Lahontan.—2 593 ft. of holes; 1 174 sacks, say, 294 bbl. = 0.113 bbl. per ft.

In his final Estacada report, Mr. Fisher says: "The holes put down with the diamond drills cost about one-third more per foot than with the shot drills."

The average rate of drilling at Estacada was 1.3 ft. per hour and at Lahontan about 0.75 ft. per hour.

The use of percussion drills for grout holes after all necessary cores are taken, as has been suggested, should materially reduce the total drilling cost on a large job, and should also reduce the time element, which is of prime importance in most river work.

The writer has information concerning other successful grouting in dam foundations in a foreign development, but, unfortunately, is not at liberty to communicate it at this time. Although the method described has inherent possibilities possessed by no other system, cases may arise wherein the impregnation of an extended rock mass with grout is not necessary, but where a definite and absolute sub-surface cut-off is required. There are many difficulties and limitations incident to the construction of a cut-off wall in an excavated trench for this purpose, especially for work under a high head where the desired depth is great, and, in connection with the consideration of the grouting scheme, certain alternatives for such limited applications have been developed. No opportunity has occurred for testing these, but some suggestions concerning them appear to be pertinent to this discussion, and may lead to the further development of foundation treatment methods.

Fig. 16 illustrates the general features of a suggested cut-off; this plan contemplates drilling a single line of 6-in. holes on 9-in. centers, broaching the intervening webs so as to form a continuous slot, introducing interlocking steel piling to ensure a water-tight barrier to the passage of water, and firmly securing this steel curtain in place and backing it by concrete, in the manner shown. The introduction of this curtain would afford the assurance of a positive stop to the percolation of water, but would not provide for the solidification of the mass of the foundation so as to increase its bearing value. For this reason this method might be specially applicable to a stratified foundation where approximately horizontal seams or bedding planes containing clay or soft material constitute a source of danger only when subjected to erosion.

Mr. Rippey.

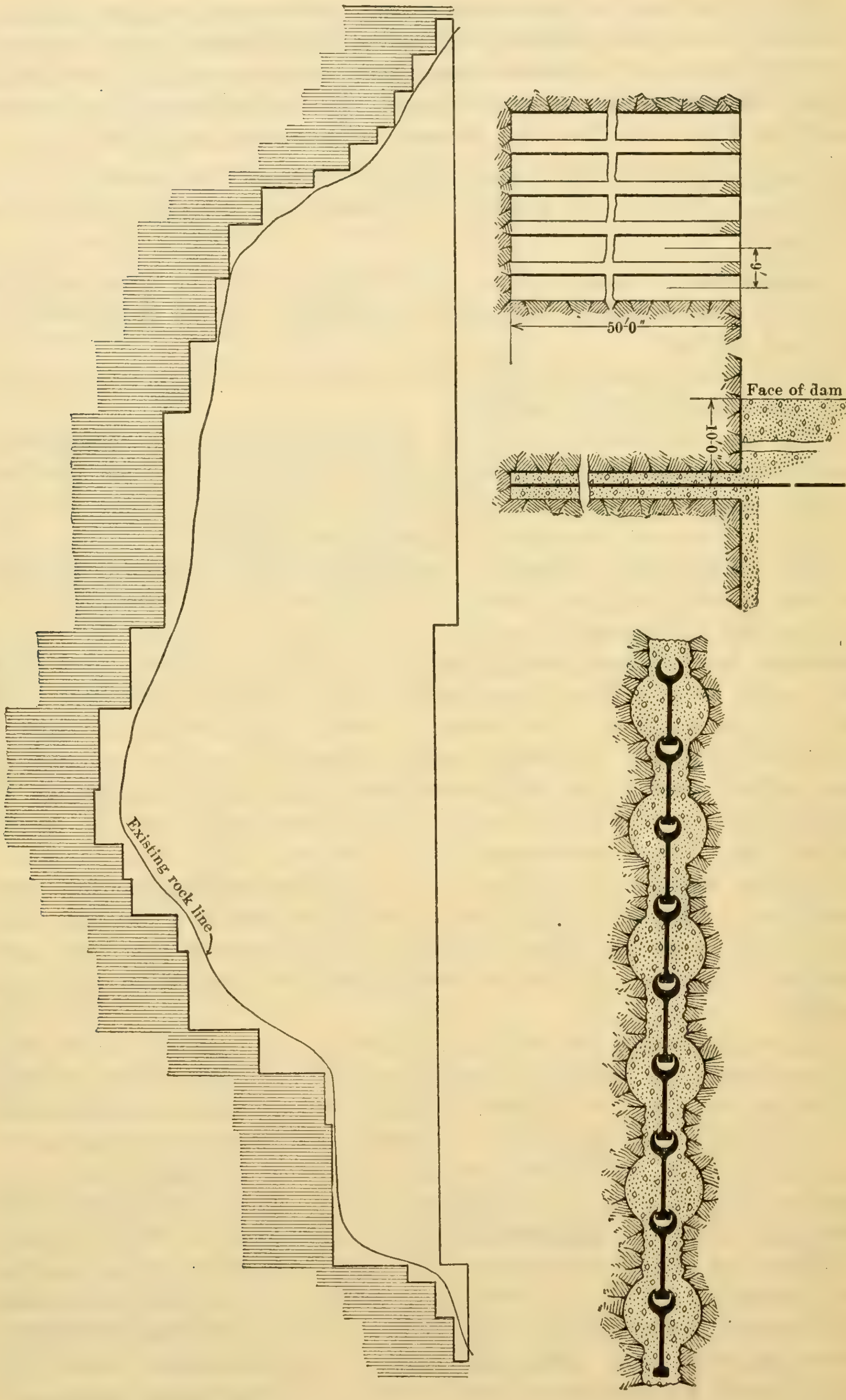

FIG. 16.

In considering the proposed method of constructing an effective cut-off barrier of this type, the details of execution remain to be studied. There appears to be no doubt that any one of several makes of percussion drills is capable of drilling 6-in. holes 50 ft. deep at relatively high rates of speed, but in ordinary work the question as to whether such holes are straight and truly plumb has been given little consideration. In the proposed scheme, the necessity for inter-connecting the drilled holes by broaching or blasting out the intervening webs to form a continuous slot, necessitates a certain degree of parallelism of the adjacent holes, and, as such drilling is without precedent for holes of this depth, the drill manufacturers have naturally been unable to make any guaranties as to performance in this respect, but there appears to be no doubt as to the possibility of working out this feature of the problem. Incidentally, the manufacturers have nothing to offer in the way of a machine for directly channeling such a slot as required, and the slot seems to be physically, as well as in the conception of method, the natural development of a series of holes drilled reasonably close together. Mr. Rippey.

As alternatives to the plan proposed, wherein the steel curtain provides the absolute cut-off within the limits of its depth, two modified schemes have been considered:

(a) Drilling a row of 6-in. holes, A, B, C, etc., on 9-in. centers, as shown by Fig. 17, Sketch 1, filling these holes solid with cement mortar or grout, and, after setting, drilling intermediate holes, 1, 2, 3, etc., which in turn would be grouted, thus forming a continuous concrete wall of 6-in. maximum thickness, if the holes are all plumb and parallel. There would be no way of determining, however, to what extent the concrete fills the spaces, and the degree of its impermeability when set, so that there would be no proof of having accomplished the desired results.

(b) Drilling a single line of 6-in. holes, A, B, C, etc., as before and forcing grout into them, which would be expected to permeate fully the intervening rock, as indicated on Fig. 17, Sketch 2.

The assurance of impermeability with this procedure would be even less than in plan (a) and both ideas were discarded as insufficient.

Inasmuch as the creation of a continuous slot 50 ft. deep is a novel proposition, the execution of the programme would require the most careful supervision, in order that detail methods might be devised which would insure the complete realization of expectations at minimum cost. In the first place, the drill should be mounted on a frame constructed so as to be easily and quickly moved from hole to hole and maintained plumb under the powerful high-speed stroke contemplated, to the end that the holes should be as true and plumb as is practicable. When the holes in a given section are completed, the

Mr. Rippey. broaching of the intervening webs of rock may be effected by replacing the 6-in. drilling bit with a broaching bit and disconnecting the rotating devices used for driving round holes, so that the bit may be partly rotated by hand, as required by the varying conditions

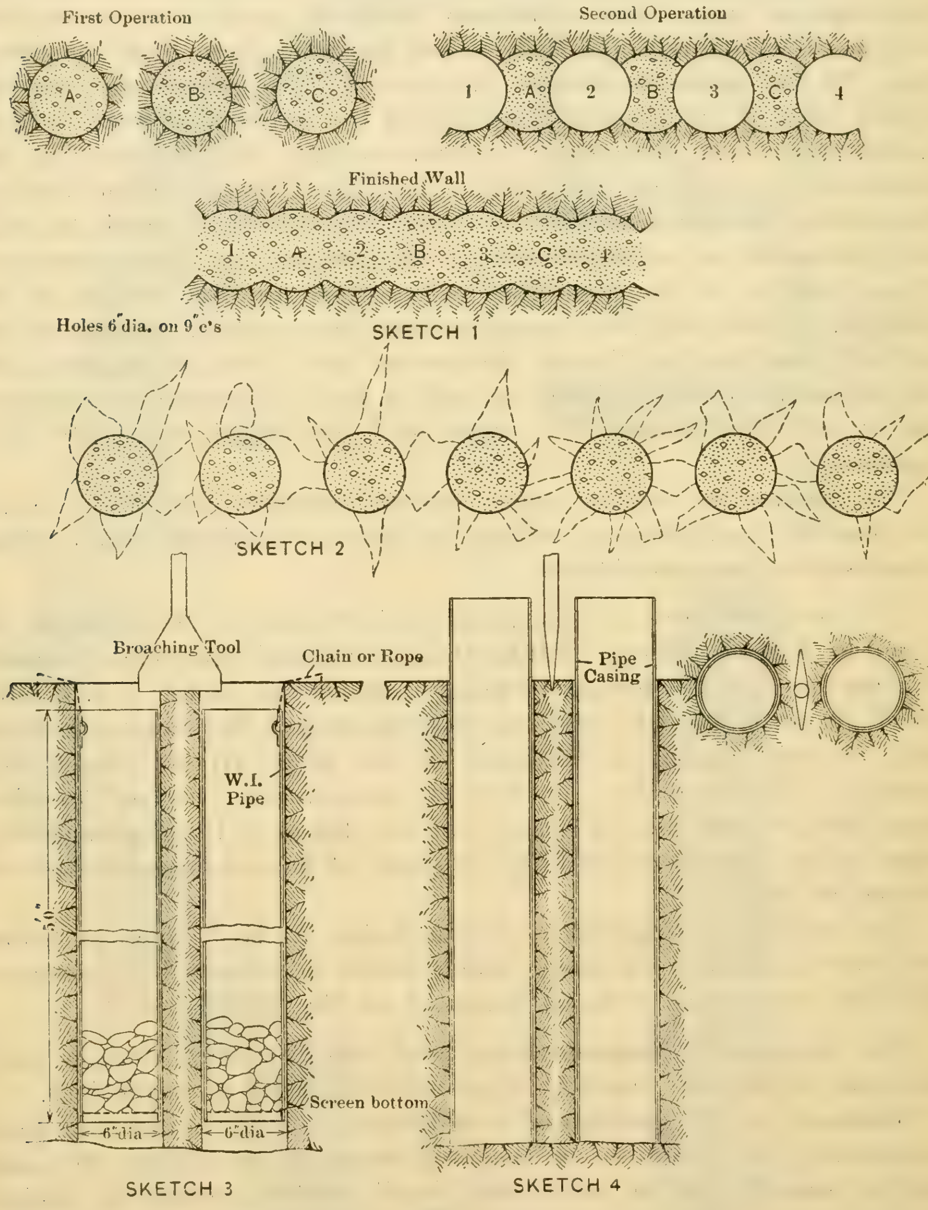

FIG. 17.

encountered. It seems possible that in some instances the original holes might deviate from the theoretical lines sufficiently to warrant drilling additional intermediate holes in places to facilitate the broaching, and it also appears possible that a limited amount of blasting might be justified in order to break out irregular webs into the con-

tinuous slot; these, however, are details to be worked out in the field during the progress of the work. Mr. Rippey.

In drilling the original holes there should be no difficulty in washing the cuttings to the surface in the usual manner, so that each hole on completion should be practically clean. There is opportunity for some ingenuity in experimenting with the broaching process, both as to the cutting and the removal of the cuttings, and it is obviously important to be assured that these cuttings are removed so as to interpose no obstacle to the introduction of piling or the complete filling of the slot by solid concrete. This cleaning out of the broach-

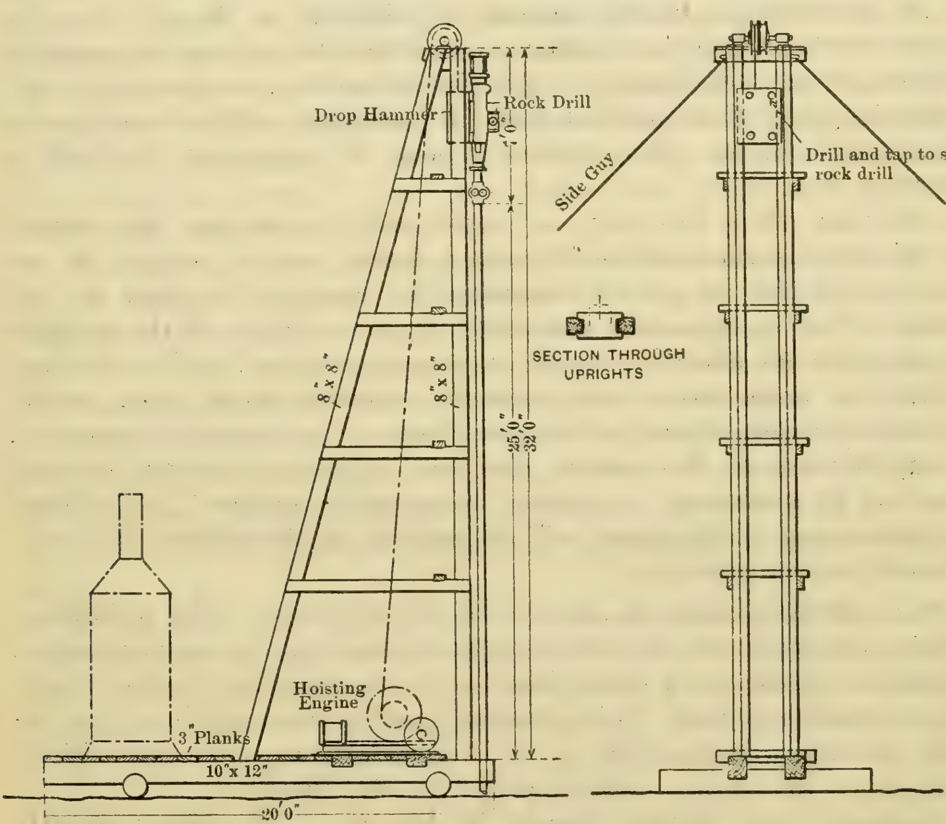

FIG. 18.

ings may require some experimentation to secure the best results, but one method which suggests itself is illustrated in Fig. 17, Sketch 3. Two standard wrought-iron pipes, having an outside diameter of $5\frac{9}{16}$ in. and perhaps 5 ft. long, are indicated, with the lower ends covered by a close-meshed wire netting which will pass water, but not cuttings. These tubular buckets, supported by chains with their tops just below the lower limit of stroke of the broaching bit, would collect almost all the cuttings, being raised and emptied when full. This would leave about 5 ft. at the bottom of the holes which could not be treated in this manner, but a shorter pair of buckets might be used at the

Mr. Rippey. bottom, and the final cuttings might be washed out as completely as possible.

Another method considered, although probably more expensive and less favorable for the rapid action of the broaching bit, is illustrated in Fig. 17, Sketch 4. This involves placing temporary pipe casings in two adjacent drilled holes to guide the broaching tool and prevent its cuttings from dropping into the clean, drilled holes. In this plan the broachings from the web would be washed to the surface by a water jet in the same manner as in the drilled holes; a few trials should determine the most effective method.

As indicated, a limited amount of blasting in the slot may be found expedient, and there is also a possibility that a heavy rectangular tool might be used finally, in pile-driver fashion, to break out any remaining projections and establish the required width of continuous slot to the bottom. The general scheme of mounting the drill is shown by Fig. 18.

Although there has been no opportunity to develop the curtain cut-off, no insurmountable difficulties appear, and it should be entirely practicable to perfect apparatus for creating the deep slot required. This should assist materially in the solution of the problem of providing an effective cut-off in general practice; and convenient commercial apparatus of the character required might thus greatly facilitate the development of certain classes of water-power properties.

In all work of this nature the most faithful attention to every detail of its execution is essential to complete success. It is hoped that discussion of this paper will be general, as the subject is of considerable current interest.

Mr. Hulse. S. C. Hulse, Assoc. M. Am. Soc. C. E (by letter).—The solidifying of dam foundations by the introduction of cement grout under pressure, through drill holes, is a rather large problem, which has hardly, as yet, been thoroughly tested. This statement applies particularly to the volcanic formation encountered in the Cascade Range and dealt with at Estacada in the work so ably described by Mr. Rands.

Inasmuch as S. Howard Rippey, M. Am. Soc. C. E., who is properly accredited by Mr. Rands as the originator of the plan to grout a cut-off curtain across the (Second Clackamas) site several miles above Estacada, will doubtless take part in the discussion of this paper, the writer will confine himself rather to certain practical aspects of the grouting problem.

When the writer went to Oregon for Mr. Rippey in the fall of 1909, he had been given clearly to understand that his work at the Second Clackamas was to be very largely of an experimental nature. Mr. Rippey dwelt at length on the fact that his ideas (as generally expressed in the extract from the paper by Mr. Schreiber on page 10*

* *Proceedings*, Am. Soc. C. E., for January, 1914.

of the paper) dealt with something yet to be proven, and he, in spite of his strong belief in the ultimate result, insisted that his own attitude toward the results of the experiments to be made, would be most conservative. Mr. Hulse.

The experiments at the Second Clackamas were halted (by the postponement of the work for business reasons) before any positive results were obtained, as regards the actual efficiency of grouting. Somewhat more than 5 000 lin. ft. of core-drill holes were put down, and these ranged from 40 to 250 ft. in depth. Many and exhaustive pressure tests were made in these holes, and they ranged, from a few which were absolutely tight against 200 lb. pressure of water, to those which took water freely at almost no pressure. Often the holes were intercommunicating and sometimes the water showed on the surface of the adjoining territory—although this did not often occur. In many cases it was impossible to discover what became of the water that some of the looser holes took freely, although very great effort was made to trace it.

After many delays and disappointments, a grouting machine was landed on the work, and experiments were begun, but, before these had proceeded to any very definite result, the machine was ordered to Estacada where things were going with such a rush as to overshadow the necessity for further investigation at the upper site.

About all that was learned at the Second Clackamas, as regards actual grouting operations in breccia, was that a hurried introduction of grout, in the proportions of from 1 of cement to 1½ to 7 parts of water, did not penetrate the breccia sufficiently to make a properly tight job.

It was also demonstrated that air-mixed grout is very deceptive, and that the appearance of smoothness which it speedily assumes, while the air bubbles up through it, is quite likely to be a cloak for innumerable balls of dry cement of varying sizes, which have formed as the cement struck the water and are not broken up by the air in any reasonable length of time, as would be done by the mechanical action of a paddle mixer.

The openings in breccia range from the tiniest of pores into which cement may only be forced, if at all, with the greatest care, but through which water under high pressure will find its way, to cracks and cavities of considerable size. Any one who has had experience in cleaning the valves of a grout machine which has become plugged with a ball of dry cement under pressure may readily realize the facility with which badly mixed grout may close entrances to openings and thus prevent them from being properly filled.

The writer's experience with surface leaks at the upper site had not been encouraging, up to the time the experiments were discon-

Mr. Hulse. tinued, but, on subsequent work, he has learned how to handle these, as will be detailed later.

To proceed to the conclusions arrived at on page 36* of the paper:

(1) The advisability of a concrete cut-off wall under the foundation of a dam is open to serious question, and the writer has heard more than one able and experienced engineer declare against the practice. To put in such a cut-off, a trench must first be opened. If this is done by hand, in any material capable of bearing a great dam, the work is likely to be slow and costly. If it is done by blasting, and particularly in breccia, where the use of a channeler is said to be impracticable because of the nature of the material, there is great danger of doing almost irreparable damage to the surrounding material, and this is exactly what happened in the left channel at Estacada, where the breccia adjoining the heel-trench was shaken and cracked in all directions by the blasting. The writer believes that grouting, properly done, may be used as a satisfactory substitute for a concrete cut-off, and to the top of the foundation rock.

(2) The testing of drill holes is such a fascinating pursuit (and the writer speaks from experience) that it may easily be carried beyond the limits of practical usefulness. However, the tank method is quite preferable where much testing is to be done.

(3) This method of procedure might readily entail an almost prohibitive amount of shifting about, and, if carried to its logical conclusion, might even lead to a complete and wholly unwarranted interruption of the grout and drill work. More will be said about this in connection with Conclusion (5).

(4) This conclusion is very true, and emphasizes the desirability of a grouting machine which may be operated by water pressure instead of by air. The following of water into the hole, behind the grout, would not be objectionable, whereas the air is a nuisance, and has to be closely watched, in this respect. The writer has hopes of developing such a grouting machine.

(5) and (3) There are two sorts of grouting which may be accomplished: plugging, and sedimentation or silting. There are several ways of going after subterranean territory, through the medium of a drill hole, in the hope of making that territory impervious to the passage of water under pressure, and the writer wishes now to present the method which seems to him best suited to the penetration of every opening of sufficient size to admit cement particles in the form of grout. This method is practically irrespective of the nature of the rock to be grouted.

The first necessity is a reliable driller, who realizes his responsibilities and will enter into the spirit of things. Fortunately, such drillers are not the exception, although one does occasionally meet with

* *Proceedings*, Am. Soc. C. E., for January, 1914.

an experience such as has been mentioned by Mr. Rands in connection with his search for cement in the cores. Mr. Hulse.

Casings should be set beyond the possibility of their coming loose. If this is impossible—as is sometimes the case in soft breccia—a mat of concrete, preferably to be incorporated later in the main structure, may be laid on top of and well bonded to the rock surface, at or near the heel line, and the casings set in this mat. The mat should be as narrow as possible, so that it will not close any more surface leaks than necessary.

If a core drill is used, the loss of drill water or the striking of a seam or crevice is the best indication of the time to test the hole. If a percussion drill is used, it may be well to test at stated depths—say, every 10 ft.—because the evidence of what is encountered by a percussion drill is not nearly so reliable as in the case of a core drill.

The point is that leaks should be grouted as soon as encountered. This is for the purpose of closing openings which would let the pressure out of the lower portions of the hole (yet to be drilled) were they not closed. In the absence of special equipment for testing purposes, charges of air or water blown through the grouting machine will answer perfectly well to indicate how freely the hole may be expected to take grout, and to show up any leaks on the surface.

The first grout introduced should be quite thin—say, 1 part of cement to 30 parts of water—and it should be absolutely free from lumps of any sort. Pressure should be applied gradually, and the instant a surface leak shows, the pressure should be dropped to such a point that only water (perhaps colored with cement) flows from the leak. This is the starting point for a successful closure of that leak by a process of sedimentation, as contrasted with the possibility of plugging it immediately by using thick or lumpy grout. From this time forward it is a matter of patience and of judgment in the application of pressure and the thickening of the mixture, and a great deal of patience is worth a very little thickening, as judged by the final results. Cement is never wasted by this process of "slow grouting" as long as it does not appear at the surface, and a careful nursing of the pressure will deposit the cement underground where it is wanted. In time—sometimes very quickly—the leak fills, and, as the resistance increases, more pressure is applied, until finally the hole refuses at the maximum pressure available. An exploration of many surface leaks thus "slow grouted" under the writer's direction, has disclosed the fact that they had been tightly filled with cement, and so very tightly as to preclude any subsequent opening due to shrinkage, as would have been the case had they been merely filled with the thickest grout that could have been run through them.

There is nothing in this sort of treatment to preclude the almost immediate washing out of the drill hole (after refusal) and the con-

Mr. Hulse. tinuance of the drilling to the next point where grouting may seem advisable. Holes grouted with thick grout will frequently take more grout, if re-tested within a few hours after the first refusal at maximum pressure, but it is very exceptional when one succeeds in making any impression whatsoever in re-testing a hole which has been "slow grouted" as just set forth. In the first case, the openings are plugged with a mixture which subsequently shrinks; in the second case, they are silted full so tightly as to preclude shrinkage.

As an illustration of the results to be gained by patience, the writer once grouted for eight continuous days and nights on a hole that took somewhat more than 200 tons of cement. For several days, thin grout was literally poured into that hole—much of it by gravity—and the grout was kept thin until the hole showed signs of closing up. Had Conclusion (5), as stated by the author, been followed, the operation would doubtless have ended much sooner, but the result (the hole, thus far, has proved to be tight against a head of 130 ft.) might have been quite different. Incidentally, of the more than 200 tons of cement used, perhaps a dozen sacks were wasted through the leaks which developed—surely not a high price to pay for placing where it was wanted the rest of the quantity used.

There is good evidence that Portland cement sets very slowly—if at all—after it has been introduced underground in this fashion, but, should the process of grouting necessarily entail an assumption of the necessity for the setting of the cement used? Rather should it not be looked on as an enforced closing of openings or passages by the silting or sedimentation of an insoluble material, introduced under an ultimate pressure greater than any which might subsequently tend to dislodge it? If the cement does set up, so much the better.

If the suggestion of the author, in connection with the grouting done at Estacada, that: "* * * so far as this job is concerned, that over a portion of the cut-off no grouting was needed, and that over those parts where it was needed it did little good", is to be sustained, what of the 1 920 bbl. of cement that were used? It is hardly to be assumed that all this cement went to close the cracks opened by the blasting of the heel-trench and to waste in dealing with surface leaks. That portion of the 1 920 bbl. which was not used for these two purposes must have gone somewhere in the foundation and have closed openings which would much better be closed than left open—else, why grout, or why bother about cut-offs? The action of the two test holes which were drilled inside the dam and flowed water, as set forth by the author, indicate obviously enough that the grout as applied did not close the breccia tightly, or, that the water passed under the curtain and then rose; and the writer is strongly inclined to the former view of the matter, but does not feel that the evidence in the case is to be viewed as damaging to the grouting system of tighten-

Mr. Hulse.

ing foundations. Rather, he believes that the Estacada work shows remarkable results in favor of the system. We have here, in the beginning, a case of a conservatively worked out scheme which is to be based on experiment and subject to experimental proof. Before anything much has been learned from the experiments, they are interrupted and a practical application is made, and under circumstances by no means favorable to a fair trial. Aside from the blast-shattered area which complicated the situation, the grouting work at Estacada was continually interfered with and hustled about, and hurried beyond the possibility of proper conduct, by the exigencies of rush construction; and, in the opinion of the writer, the engineers in charge of the grouting did remarkably good work under the circumstances. If, as the author states, no leaks of consequence have been observed below the Estacada Dam, may not a certain amount of credit be given to that part of the 1 920 bbl. of cement which was not wasted in handling surface leaks, and is it not conceivable that, had the work been less rushed, and had greater care with the grouting been possible, the flow in the test holes inside the dam might have been less?

Grout work is likely to be at a disadvantage when brought into conflict with the man whose sole idea of the conduct of work is to set a new record for depositing a given quantity of concrete in the shortest possible time. For the grouting, there is little in evidence but a few small pipes—which may or may not connect with an underground opening which might, later, imperil the integrity of the concrete man's work, when the reservoir fills—and these small, insignificant pipes may even get in the road of the concrete man's buckets, and otherwise become very much of a nuisance.

If, therefore, it is possible to do the grouting and get the pipes out of the way before the beginning of actual construction, so much the better. If the grouting and construction must be carried on simultaneously, every effort should be made to keep them from interfering with each other; but, in case they must conflict, it seems rather superfluous to point out that, inasmuch as the integrity of the whole job will depend to a very great extent on the success of the grout work, the latter should be given at least "an even break" in the matter of precedence. An open cut-off trench, by its very presence, compels respect and consideration, but a line of 2-in. grout pipes, which may be expected to accomplish cheaper and better results than the cut-off trench, must frequently be aggressively guarded by the man in charge of them.

In any case, it is desirable that provision be made for drilling and grouting after the completion of the work, should later developments show the need for this later work. Usually, this may be done more at the cost of foresight than at the cost of trouble and expense.

The author's suggestion about the drilling of a row of holes close

Mr. Hulse. together and then broaching them into an open slot which may then be grouted, is open to the objections against blasting in the foundation material under the heel of a dam to be. If, however, this might be accomplished without damage, the writer would suggest that, before the holes are broached, they should first be carefully grouted so as to silt up the adjoining territory, and thus reinforce the thin solid cut-off to be made by filling the slot. Further, it would be most desirable, in filling the slot, to do so by introducing the grout at or near the bottom of the slot, and forcing it to rise slowly toward the surface. A succession of such operations, to be accomplished in a deep slot, by a gradual raising of the grouting pipes, would probably insure the tightest filling of the slot with the least chance of subsequent shrinkage and cracking of the material deposited therein. Without some reinforcement, which might be very difficult to place in a thin wall like this, the writer would not consider the scheme except in connection with a careful silting up of adjoining territory by first grouting the holes, as has been stated.

As a general proposition, the writer believes that the silting up of a foundation by grouting and by the action of the water impounded behind the dam, offers possibilities distinctly preferable to those of the conventional cut-off wall. In the case of a clear stream, like the Clackamas, it is usually practicable to make the water muddy by dumping or sluicing into it material from the banks, while the reservoir is filling, and thus close many or all of the openings missed by, or too fine for, the grout. The practice of shooting into foundation material under the heel of a dam is more than likely to make a lot of trouble—witness the left channel at Estacada—and the penetrating qualities of cement grout, properly handled, are very great. If the rock is good, why replace it with a concrete cut-off? If the rock is bad, and yet so solid as to necessitate blasting for its removal, how much better to silt it tight and leave it undisturbed?

Mr. Fisher. FRANK R. FISHER, ESQ. (by letter).—In the writer's final report as Resident Engineer in charge of the construction of the Estacada Dam, he stated, in discussing the grouting of foundations, that an impervious cut-off was not accomplished, that is, to the absolute degree had in view when the method was adopted. The varying and erratic nature of the formation made the solution of the problem largely experimental and one that could only be worked out by a thorough trial and without any positive assurance as to what the final outcome would be.

The writer does not consider that the method should be condemned or judged by the failure to obtain absolute tightness in the very peculiar material with which the engineers had to deal, but rather that credit should be given them for reducing the possibility of seepage to the extent that practically no uncertainty regarding the

safety of the foundations in that respect would remain. That this was accomplished, and the foundation material benefited by the treatment, he believes an analysis of the results, as stated in his report, will show. Mr. Fisher.

Mr. Rands arrives at the following conclusion: "that over a portion of the cut-off no grouting was needed, and that over other parts where it was needed it did little good." The first part of his statement is correct, in that over a considerable portion of the right channel and island sections, the average formation was found to be in good condition, not absolutely tight, but developing only a low rate of seepage under test. Naturally, this evidence was not apparent until it was disclosed eventually by the progress of the work, and then, in view of the ever-present uncertainty, due to the fact that occasional points would develop unexpected weakness, it was deemed advisable to follow out the original plan to completion. In the light of final knowledge acquired from the complete data obtained, it was evident that the grouting could have been omitted over certain stretches, and the number of holes on these sections materially reduced.

The remaining section, that across the left channel, is evidently what Mr. Rands refers to as: "other parts where it was needed it did little good". There was no question as to the need of some kind of treatment in this particular locality, as the material, from the standpoint of permeability, was the worst met with on the site of the dam. The water-pressure tests gave high rates of seepage, communication existed between holes located over a considerable distance, and also with the surface as many as 70 ft. from the point of testing. The holes took grout freely and, in the aggregate, several hundred barrels of cement were forced into the seams and crevices existing throughout the rock. As the individual seams were apparently of little width, the large quantity of grout taken would seem to indicate that it was diffused over a wide area.

Although the tests on the final proving holes, at the full depth of 50 ft., developed considerable seepage, it was very much less than the original. The most encouraging feature, however, was the very satisfactory results obtained for the first 30 ft. Over a length of cut-off of 70 ft., the aggregate of the seepage through thirteen holes, spaced approximately 6 ft. apart, when tested under a hydrostatic head slightly in excess of that created by the dam, was only 0.2 sec-ft., and it should be noted that this seepage was obtained by boring into the vitals of the foundations, and opening up and subjecting to direct pressure all points of weakness intercepted by the thirteen holes. It should also be kept in mind that this test was made on what was originally the weakest section of the entire cut-off.

It is also obvious that the test conditions were much more severe than could occur normally from the pond pressure, and, therefore, it

Mr. Fisher. is reasonable to conclude that the grouted cut-off is effective for a depth of 30 ft., and as this is reckoned from the bottom of an overlying 10-ft. concrete wall, the total effective cut-off is 40 ft.

As to the choice between a concrete and grouted cut-off for this depth, it is only necessary to point out that the high cost, difficulties of construction, and the element of time required—of vital importance in this case—would practically prohibit the use of concrete.

Evidence of the satisfactory final condition of the foundations was furnished subsequent to the completion of the dam and the filling of the pond by a thorough examination of the surface of the rock inside the dam, and the unwatered bed of the river immediately down stream. No springs or seepage could be observed, with the exception of a few very slight indications along the side of the left bank, and it is more than likely that these were from surface drainage.

The foregoing facts may not offer indisputable proof that the final satisfactory condition of the foundations was entirely due to the merits of the grouting treatment, but the evidence is in its favor, and it is entitled to all the credit that the results seem to warrant.

Mr. Coburn. H. L. Coburn, M. Am. Soc. C. E. (by letter).—Mr. Rands is to be congratulated on the thorough manner in which he has presented the problem and attempted solution in the matter of the cut-off under the Estacada Dam. The information contained in this paper will be of great interest to the Profession. There is little which can be added to Mr. Rands' statements, and the writer thoroughly agrees with his conclusions as to this particular structure.

In such a foundation as was found in Estacada it seems that the only reason for making extraordinary efforts to secure an impervious cut-off at great depth would be for the purpose of saving water, and that it was not at all necessary to prevent undermining, as the rate of flow through this foundation was so slow as to be negligible. If, however, some further protection was deemed necessary, the writer believes that the concrete cut-off could have been carried very considerably deeper than was done for a lesser sum than was expended on the grouting, and with much more definite assurance of success. If, in addition to this concrete cut-off, a carpet of fine silt, either of clay or volcanic ash or other fine material—which is present in large quantities—had been sluiced into the river bed and against the dam and extending several hundred feet up stream, it would appear certain that a very satisfactory seal would have been obtained. Indeed, the writer is confident that the natural deposit of silt which is inevitable in such a pond will seal this structure very completely, and in a comparatively short time.

Mr. Cushman. William H. Cushman, M. Am. Soc. C. E. (by letter).—The general scheme had been adopted and some of the earlier drilling accom-

Mr. Cushman.

plished at the time the writer took general charge of the project. The execution of this portion of the work, as already determined, became a duty, regardless of an individual lack of confidence in the ultimate successful accomplishment of the end in view.

That the foundation material was sufficiently good to carry all the pressure to which it could possibly be subjected, will be conceded by all engineers familiar with the designs for the dam and the foundation material.

Although the breccia was porous, it was substantially free from open cracks, and it was scarcely to be considered a case where the percolation of water through the foundation material would result in undermining the dam.

Admitting these two propositions to be true, it would seem that the only practical consideration would be the one mentioned on pages 35 and 36* of the paper, that is, whether the water lost by percolation would be valuable enough to justify the cost of the grouting work.

Although the Clackamas is a clear-water stream, undoubtedly some silt depositing could be depended on, and possibly selected materials could have been deposited in the pool above the dam to assist materially in silting up the porous material.

Throughout the paper (as mentioned on page 20, the seepage test detailed on page 22, and again on page 28,* where serious thought of discontinuing grouting was entertained), considerable skepticism is expressed as to the effectiveness of the method of treatment. The quoted opinion of the author, that "Over those parts where it was needed it did little good", will be commonly agreed with, and cause little surprise to those who have had similar problems to solve.

The suggestion mentioned on page 37,* that the rock between the rows of drill holes be shattered with small charges of powder, originated with the writer, and it still seems that such shattering might have facilitated the flow of "thickened grout" and made it possible to restrict the treated area to the immediate vicinity of the cut-off wall, rather than diffuse it to remote points by subjecting it to the heavier pressure required to force it through small openings.

It would be interesting to know whether an attempt has ever been made to shatter a trench through rock, to a depth of as much as 50 ft., and then introduce grout or thin mortar (fed through pipes from the bottom and flowing upward or otherwise) in such manner as to produce, in effect, a concrete wall to the depth stated. It would not seem impracticable to accomplish this, and do away with surface capping with concrete and the use of excessive pressure.

Considerable thought and investigation were given by Messrs. Rippey, Fitzgerald, Schreiber, and the writer, to various methods of cut-

* *Proceedings*, Am. Soc. C. E., for January, 1914.

Mr. Cushman. ting an open trench or seam through the rock to the required depth, in order that an impervious curtain could be introduced. Special channeling machines, drill holes closely spaced and then broaching out the material between holes, and even open cut, were considered. It would be of interest to hear from engineers who have accomplished something along these or similar lines.

Mr. Rands is to be commended for the painstaking manner in which he has presented this interesting subject, and, though it be admitted that the procedure was, to a large extent, futile, nevertheless, his subsequent work has proven the value of this first experiment.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ERNEST PONTZEN, Cor. M. Am. Soc. C. E.*

DIED OCTOBER 13TH, 1913.

Ernest Pontzen was born at Budapest, Austria-Hungary, on January 20th, 1838. He obtained his early engineering education at the Polytechnic School of Vienna. In 1856, he went to Paris and entered the French engineering school, l'Ecole Nationale des Ponts et Chaussées, as an "Externe" student, and was graduated in 1860 at the head of his class.

After a study trip in England, Mr. Pontzen returned to Austria-Hungary, and filled various positions in connection with the railways and harbors of that country. He was Consulting Engineer to the Anglo-Austrian Bank for 4½ years, and, in that capacity, directed the construction of several railway lines, made a study of various railway projects, and became the Director of a railway company in Hungary.

In 1873 he made his first study trip to the United States. He made a second trip in 1876, to serve as member of the Jury in the Railway Group at the Philadelphia International Exposition. Returning to Paris, he established himself there permanently, and eventually became a naturalized French citizen. He married Miss Hirtz, daughter of the distinguished physician and professor of that name, of the University of Strassburg, who survives him.

Mr. Pontzen was the author of numerous publications dealing with the construction and operation of railways. His most important work, issued in collaboration with Mr. Lavoinne, and entitled "American Railways," was published in 1880-82, the first volume dealing with the construction and cost of railways, and the second with their operation.

Mr. Pontzen served on various important commissions. He had been a member of the Government Commission for the technical exploitation of railways since 1884, almost from its inception. Acting in this capacity, he was one of the delegation which represented the French Government at the International Conference for the promotion of technical uniformity on railways, and more recently he was a delegate of the Minister of Public Works to the International Railway Congress at Washington, and, later, to that at Berne.

* Memoir prepared by Charles L. Strobel, M. Am. Soc. C. E.

He was also selected as an expert and arbitrator in very important cases, such as the Loetschberg Tunnel, the Harbor of Varna, the acquisition of the Algerian East, and the Cé Bridge accident.

He was a Member of the Council on Instruction (Conseil d'Instruction), and of the Council on Betterment (Conseil de Perfectionnement) of l'Ecole Nationale des Ponts et Chaussées.

Mr. Pontzen was a Past-President of the Society of Colonial Engineers and Past Vice-President of the Society of Civil Engineers of France. He was one of the founders and, up to the time of his death, was President of the Friendly Society of "Externe" Graduates of l'Ecole Nationale des Ponts et Chaussées. He was also an officer of the Legion of Honor and a Commander of various French and foreign orders.

This brief outline of Mr. Pontzen's career will indicate that it was fruitful in engineering achievement and rich in well-merited honors. He went to France as a stranger, and died in the country of his adoption, having won not only the esteem of Frenchmen, but to a singularly high degree their love and affection.

Mr. Pontzen spoke French, German, English, Italian, and Hungarian, and with foreign engineers whom he met at Congresses, he was able, therefore, to converse in their mother-tongue. All who met him in this way remember him most pleasantly for his uniform courtesy and cheer. American engineers who had the good fortune to know him were always sure of a cordial welcome at his hands, and he freely gave them of his time and attention in the promotion of their interests or their comfort and pleasure. He was the kindest, most considerate, and most devoted of friends. It may be said of him that the conduct of his life was so noble and inspiring, that the highest tribute will be paid to it that can come to any man, in that it has left the desire, especially on the part of the younger men, to emulate his example as best they can.

Mr. Pontzen was elected a Corresponding Member of the American Society of Civil Engineers on January 5th, 1876.

ARTHUR LINCOLN ADAMS, M. Am. Soc. C. E.*

Died September 17th, 1913.

Arthur Lincoln Adams, son of Jacob Clendennin and Nancy McCoy (Hamilton) Adams, was born on September 15th, 1864, on a farm near Greensburg, Ind. He was a lineal descendant of John Adams, who came to Plymouth Colony, Massachusetts, in the ship, *Fortune*, in 1621.

* Memoir prepared by Franklin Riffle and W. A. Cattell, Members, Am. Soc. C. E., and A. Kempkey, Assoc. M. Am. Soc. C. E.

When about eight years of age he moved with his parents to Topeka, Kans., where the family resided for a period of about three years, and then returned to Indiana. In 1882, soon after the death of Mr. Adams' father, the widow again moved with her children to Topeka, where she resided during the remainder of her life.

In his early boyhood Mr. Adams attended the district schools of Indiana, and the public schools of Topeka. Later, he entered the Greensburg High School, from which he was graduated in 1882. He was a student at Hanover College, Ind., in 1882-83, and Washburn College, Topeka, Kans., in 1883-84.

During the summer of 1883 he was in the employ of the County Surveyor of Shawnee County, Kansas, and assisted in surveying in and about Topeka. During the following summer he was with a surveying party engaged in subdividing Government land in Southwestern Kansas. Starting as a Chainman, he was promoted to Transitman before the close of his engagement. When the survey was completed, each member of the party was requested to state, under oath, that the work had been done correctly, to the best of his knowledge and belief. Being somewhat in doubt as to the accuracy of the survey, Mr. Adams, together with several others of the party, strenuously objected to being sworn. At this juncture, one of the youthful mutineers made the discovery that Quakers were permitted to affirm in lieu of taking oath, whereupon Adams insisted that the same privilege be accorded him and his conscientious associates. His arguments prevailed, the affirmations were duly made, and all the members of the party were given honorable discharges.

In the fall of 1883 students of the State University of Kansas started a movement to organize a State oratorical association, to affiliate with the Interstate Association. Mr. Adams was a delegate from Washburn College to the constitutional convention. He took a prominent part in the proceedings, which resulted in the organization of a State association, and was active in arranging for the first contest. He participated in the trial and settlement of a famous case of plagiarism which developed in that contest.

In 1884 he became a student in the Engineering Department of Kansas State University, and was graduated in 1886. At a public rhetorical recital he read a humorous essay entitled "The Philosophy of Selfishness", in which his theories and conclusions were so extreme as to create a sensation among his hearers. Only a few of the audience appreciated the undercurrent of humor which pervaded the essay, or realized that the extreme views advocated by the speaker were purely for rhetorical effect.

During the spring of 1886, he obtained employment in the Engineering Department of the Burlington and Missouri River Railroad, in Nebraska, in the office of one of the resident engineers, and was soon

promoted to Division Engineer on Construction. In this position he displayed unusual efficiency, originality, and judgment, for one of his age and experience.

During the summer of 1887, he concluded to try his fortune on the Pacific Coast. He accordingly accepted a position as Leveler in a location party on the Oregon Pacific Railroad. When the party was disbanded, during the following winter, he went to Southern California in search of employment and spent several months in Los Angeles.

During the spring of 1888 he returned to the Pacific Northwest to accept a position as Assistant Engineer on Construction, offered him by the Chief Engineer of the Oregon and Washington Territory Railway, now a part of the Northern Pacific System. In this position he had an excellent opportunity to observe and study the wonderful resources of Eastern Oregon and Washington. He was quick to grasp the significance of the development of this vast inland empire, and to see with prophetic eye the varied and multitudinous engineering problems to which this development would give rise. Many thriving towns were in various stages of development, and some of them had already reached a point where the necessity for sewer and water systems was occupying the attention of their citizens.

Up to this time, Mr. Adams, like many other young engineers, had followed the lines of least resistance. He had become a railroad engineer through force of circumstances. After graduation he had accepted a position on a railroad because it was the only position immediately available. Municipal engineering, however, was his choice, and he now determined to make a start in this field. The Town of Pendleton, Ore., had recently created the office of "City" Engineer, and the Mayor was given the appointing power. Before resigning his position as railroad engineer, Mr. Adams discussed the matter with the Chief Engineer, who accepted his resignation with regret, but, nevertheless, co-operated with him in his efforts to secure the coveted municipal position. Mr. Adams was successful in obtaining the appointment, at the same salary he had received from the railroad. In conducting the negotiations which terminated so happily, he exhibited that remarkable business sagacity which contributed so largely to his later successes. Foreseeing that as "City" Engineer of a small town his duties would not be very onerous, he stipulated that he should be granted the privilege of performing such private work as he might see fit to undertake during his term of office. He prepared a contract fully covering this phase of the question, and it was duly signed by the contracting parties. It was not long, however, before his private practice assumed such large proportions that he found it advisable to resign.

In 1890 Mr. Adams prepared plans and specifications for a system of water-works for the Town of Dayton, Wash., and superintended its

construction. During the following year he performed a similar service for Colfax, Wash.

In the summer of 1891 he entered into partnership with his brother-in-law, Robert C. Gemmell, M. Am. Soc. C. E., under the firm name of Adams and Gemmell. The new firm designed and superintended the construction of water-works for La Grande, Ore., and Waitsburg, Wash. They also designed a water system for Heppner, Ore., and a water-power plant at Kalama, Wash., neither of which was constructed, however, owing to the financial panic of 1893. The firm did considerable miscellaneous surveying in Oregon and Washington, including two large irrigation canals on the Umatilla River, Oregon, one on the Walla Walla River, Washington, and one on Eagle Creek, near Baker City, Ore. The construction of these canals was indefinitely postponed by the "panic". They prepared plans and specifications for a water-works system for Athena, Ore., and a sewer system for Montesano, Wash., both of which were constructed, although not under their supervision.

During the fall of 1893 Mr. Adams made a very complete report on an increased water supply for Astoria, Ore., and recommended the construction of a new system. His report was adopted in its entirety by the Board of Water Commissioners, and Adams and Gemmell were authorized to make the necessary surveys and plans. Before the completion of the preliminary surveys, the partnership was dissolved, and the work was continued by Mr. Adams, who, as Chief Engineer, carried it through to a successful termination—less than 3 years after having made his preliminary report. An admirable description of this work was written by Mr. Adams, under the title, "The Astoria City Water Works".* This paper won for its author the Thomas Fitch Rowland Prize for 1897. In his paper he described quite fully an ingenious device (of his own invention) for opening and closing two fire gates from a central station; this being accomplished "by opening and closing an ordinary stop-cock at the end of a line of ordinary ¾-in. service pipe leading to a patented governor attached to each of the gates."

During the construction of the Astoria Water-Works (1895), Mr. Adams prepared a report on a new water system for Eureka, Cal., and on the character and probable cost of the private works then supplying the city. After the completion of the Astoria Water-Works, he opened a consulting office in San Francisco. During 1897 he prepared plans and specifications for the reconstruction of water-works at Centralia, Wash., and also reported on projected new works for Lakeport and Oceanside, Cal.

From 1897 to 1900, he held the dual position of Manager and Chief Engineer of the West Los Angeles Water Company and the West Side Water Company. During this period he resided in Los Angeles, where

* *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 1.

he also carried on a consulting practice. As Chief Engineer of the two Los Angeles Water Companies, he was called on to plan and construct extensions in order to keep pace with the steadily increasing demand. He constructed a 5 000 000-gal. reservoir, and a pipe line to the Soldiers' Home, Santa Monica. In connection with the pipe line, he designed an ingenious weir and gate device for maintaining a definite delivery under varying conditions of pressure.

During this period Mr. Adams filled various engagements, among which were the following:

He was a member of a commission of engineers engaged by the City of Los Angeles to determine the value of the improvements made to the water-works of that city during the 30 years' lease by the Los Angeles City Water Company; he was a member of a commission of engineers engaged by the City of Pasadena to appraise the several water-works used for the supply of that city; in conjunction with J. B. Lippincott, M. Am. Soc. C. E., he reported on sources of supply and plans for an entirely independent system of water-works for Pasadena; he developed a water supply and prepared plans for its utilization, in the irrigation of a large ranch near Corona, Cal.; he prepared a report outlining plans, and giving estimates of cost, for a combined municipal water supply and power installation for Missoula, Mont.; he prepared plans and specifications for the reconstruction of the water-works of Pendleton, Ore.; he reported on the probable effect on the water supplies of the San Gabriel River, California, of the contemplated construction of certain power plants on that stream; he prepared a report on the value of the properties of the West Los Angeles Water Company and the West Side Water Company, as an aid in effecting a sale of these properties to the City of Los Angeles; he prepared a report and gave evidence concerning the value of the properties of the Contra Costa Water Company used for supplying the City of Oakland and its vicinity; and he prepared a report and gave evidence concerning the causes of an extensive land-slide which had for some years prevented the use of the large reservoir on the west side of the Willamette River, at Portland, Ore.

In 1901 Mr. Adams became Engineer and Manager of the Contra Costa Water Company, Oakland, after having been retained as an expert in the important litigation case which had just at this time been concluded.

The case was the now famous one known as the Hart case, and was instituted to enjoin the City of Oakland from enforcing the rate ordinance enacted by the City Council. Although several other eminent engineers were engaged in the case, it was conceded by all that the clear and convincing testimony of Mr. Adams as to values, and his clear exposition of the theories of valuation, had more to do with the victory of the Company than any other one thing. The Company

was granted in effect an increase in rates amounting to 15%, and, although in later years engaged in many important legal matters, Mr. Adams often remarked that he considered this case the most important of its kind in his career.

Early in 1903, Mr. Adams resigned from the management of the Contra Costa Water Company and opened a consulting office in San Francisco, and from this time he continued steadily his consulting work. Although his practice was never such as to require a large organization, it was at all times of the highest type. Among the more important of his engagements may be cited that with the City of Victoria, B. C. His report dealt with the existing conditions and present and future needs, covering the subject in complete and exhaustive detail. Later, he prepared plans and specifications covering the then present needs, and, under his direction as Consulting Engineer, they were executed.

While still retaining his consulting practice, he was, in 1906, made Chief Engineer of the Department of Greater Water Supply of the Peoples Water Company, successors to the Contra Costa Water Company, supplying water to seven cities on the east side of San Francisco Bay. Complete preliminary studies and investigations were carried on by this organization looking to the more than doubling of the water supply of this Company. A complete comprehensive plan of future development was outlined, and construction was begun on the more important elements, but financial stringency prevented carrying out more than a small portion of this work.

Mr. Adams did a very large amount of valuation work, particularly of public utilities. Among the more important properties on which be reported may be mentioned the Peoples Water Company, the Spring Valley Water Company, the Palermo Land and Water Company, the Virginia and Gold Hill Water Company, the Petaluma Power and Water Company, and numerous others.

In 1909, he designed the irrigation system for the Patterson Ranch Company, at Patterson, Stanislaus County, California. This system provides a means for irrigating, by pumping, about 18 000 acres of land on the west side of the San Joaquin River in the San Joaquin Valley, and was at that time considered to be of unique design and the most extensive and complete system of the kind in the United States.

At the time of his death Mr. Adams was engaged in the construction of a plant similar in design to the Patterson Ranch system, but more elaborate as to details, for the irrigation of about 22 000 acres of land near Brentwood, Contra Costa County, California. He was also, at the time of his death, Consulting Engineer for the Peoples Water Company, and was engaged by this Company in important rate litigation pending in the Federal Court.

Mr. Adams was foremost among those who organized the San

Francisco Association of Members of the American Society of Civil Engineers. He was elected Treasurer of the Association for a term of 2 years, and became its third President in 1907. Immediately after the earthquake in 1906, he was instrumental in holding a series of special meetings of the Association, which resulted in a study of "The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions," by a general committee and six special committees composed of members of the Association, followed by extensive reports.* Mr. Adams was Chairman of the Committee on the Effects of the Earthquake on Water-Works Structures, and wrote the Committee's report.

Mr. Adams held a high rank both personally and professionally. He was exceedingly democratic in his ideas, very simple in his tastes, and exceptionally companionable. He possessed a keen, analytical mind that served him well in the litigation with which he was so frequently connected as an expert witness. He delighted in the legal subtleties of these cases no less than in the engineering problems involved. Although active in assisting and supporting the attorneys with whom he was associated, he always commanded the admiration and respect of the engineers and attorneys who opposed him. His ability was acknowledged and his work commended by friend and foe alike. His professional career was one of uninterrupted achievement from its beginning to its untimely end. Long before his death he had attained a foremost place in the ranks of the leading hydraulic engineers of the Pacific Coast. He possessed in a marked degree the rare combination of professional skill and business acumen which enabled him to procure for his clients the best attainable results at a minimum cost. Having early in his career established a reputation for efficiency and integrity, he thereby created a demand for his services, for which, usually, he was able to name his terms. While always fully alive to his personal interests, he never for a moment overlooked the interests of his profession. He continuously sought to uphold its dignity, and to educate the public to a proper conception of its importance as a factor in the world's development. On numerous occasions he fearlessly exacted from his clients the recognition to which he believed his profession was entitled.

Mindful of his early struggles for recognition, he strongly sympathized with young engineers, and was ever ready to extend a helping hand to those in need of assistance. It was largely the desire to aid the younger members of the profession that induced him to take a prominent part in the organization of the San Francisco Association of Members of the American Society of Civil Engineers. He believed that they, more than the older members of the Society, would be benefited by membership in this organization. During the practice of his

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* *Transactions*, Am. Soc. C. E., Vol. LIX, p. 208.

profession he gave employment to many young engineers. He invariably treated his subordinates with forbearance and tolerance, and thus secured and retained their respect, loyalty, and friendship.

Mr. Adams possessed a gift which, unfortunately, is quite rare among engineers—that of clear, terse, forceful expression. This applied to his speech no less than to his writing. Whether writing a report, explaining an engineering proposition to a client, elucidating a technical point while on the witness stand, or addressing an audience, he invariably chose his words well, and without hesitation. In argument he was convincing and rarely failed to carry his point. Whenever he took a stand for or against a proposition, he gave the impression of being master of the situation. He never allowed himself to be swerved from the course which his conscience or his judgment marked out.

His energy and ability, however, were not devoted entirely to his engineering practice. He was a zealous and efficient worker in the cause of religion and morality. He was prominent in the affairs of the church of which he was a member, and also gave much of his time and resources to the Young Men's Christian Association and Young Women's Christian Association of Oakland. The Y. M. C. A. Building, and the new First Presbyterian Church edifice, both in Oakland, are to a very large extent the results of his effective work.

Mr. Adams was a well-read man. He was especially fond of poetry. Among the poets, Robert Burns was his favorite. He occasionally wrote verse of a serious nature, although not for publication. One of the writers of this memoir has in his possession several of Mr. Adams' poems, one of which, entitled "A New Year's Revery", possesses exceptional merit.

Frank T. Oakley, M. Am. Soc. C. E., who was a classmate of Mr. Adams in the public schools of Topeka and in the State University of Kansas, and also was associated with him during his professional career, writes of his life-long friend as follows:

"He was always very tolerant of the beliefs of other people, never seeming to wish to force his opinions upon others. However, when it came to argument, few could hold their own against him. My observation leads me to believe that he always followed a thoroughly digested plan of conduct and business from which he did not allow himself to vary in principle, and I believe much of his success in life's problems was due to this fact as well as to his unusually good judgment.

"Some time over twenty years ago, while sitting in camp, he outlined to me his idea of an engineering career leading up to a consulting business. When I came to live in California, I was impressed with the similarity of his outline of a career and his experience, and the way he carried on his professional business. He always insisted on giving any undertaking his best thought, and would not be identified

with it otherwise, not sparing himself hard work; and as a consequence he was always accorded a leader's place by his associates.

"I believe that Arthur L. Adams was the finest character I have ever known, and that it is not possible for me to know how much I owe to association with him. His influence was always for good and manly things. This feature did not eliminate, or in any way curb the mischievous or boyish spirit, or the joy in sport or in a practical joke, but it did eliminate any meanness—a thing that was impossible in his nature.

"In our close association, including two years of living together, I never knew him to be gushingly confidential. There was always something of a reserve about him. Nevertheless, I never knew one in whom I could more freely confide, and who was more ready with the best and most kindly and tactful advice. I knew him best during the character-forming age, when boys are fully themselves and freely open to their playmates. Yet I never knew him to do an act or express a thought that was not in accord with his later high moral and mental attainments. Nor was he in the least prudish, nor did his associates ever think of calling him a 'goody-goody boy.'

"During our college days we frequently discussed all kinds of subjects pertaining to human life and affairs. Whether the subject was a social matter, the mathematics of gambling, or theology, freedom and frankness of thought and expression were always the same, always tending to a greater and more beneficial knowledge of the subject and, in fact, a crystallization of one's own opinions. When thinking of Arthur Adams' character, I am reminded of what Riley makes the farmer say of his friend and neighbor:

> " 'Fer the name of William Leachman and
> True Manhood's jist the same.' "

C. Derleth, Jr., M. Am. Soc. C. E., pays the following tribute to Mr. Adams, both as a man among men and an engineer:

"Through his writings, particularly on wooden stave pipe, I have known Mr. Adams by reputation at least since 1896. I did not meet him personally until 1904. But since then I feel that I have had an opportunity to know of his work and to appreciate his personality. For the last eighteen years I have worked for, or have met in various ways, a great many engineers whose names are prominent in our national societies, and these men hail from all quarters of the United States. I am convinced that among them none has combined in the highest sense, better than Mr. Adams did, all of those qualities which contribute to the professional and personal character of an engineering gentleman. One always associated with Mr. Adams the qualities of integrity, industry, wide reading, learning in fields other than engineering. He had a great command of the powers of speech. He wrote fluently. He was an engineer who used reason and who had powerful judgment. Above all, he was a man of the finest moral instincts. He was always ready to help improve the tone of engineers and to raise the standard of engineering ethics. Indeed, he applied the same qualities to wider fields and was a power for good and uplift in the community in which he lived. As more men like Mr. Adams

become identified with engineering, so in proportion will the community's respect for the profession of engineering increase."

Augustus Kempkey, Assoc. M. Am. Soc. C. E., writes of Mr. Adams as follows:

"As his Principal Assistant for some eight years immediately preceding his death, the writer came to know Mr. Adams as a most refined and courteous gentleman of the highest type, every ready to give the best that was in him to the uplift of humanity and to the assistance of those around him. His thoughts were ever on the highest plane and his judgment, particularly in engineering matters, remarkable to a degree that at times seemed little short of uncanny. His mind, working perfectly clear and along direct lines, enabled him to grasp the essentials of an engineering problem almost at once, and his solution of these essentials was equally direct and expeditious. His criticisms of and orders to his subordinates were always given in a most kind and considerate manner, and no matter what the subject, he was always ready to receive suggestions and ideas from those directly under him. In all my association with him, I never saw him either reject or accept a suggestion without being fully informed as to its significance and value; and in the case of rejection, pointing out clearly his reasons therefor, and if accepting, giving due praise and weight to the value thereof. In short, he was a man who inspired and encouraged ideas and suggestions from his subordinates, while withal he never provoked that familiarity which ultimately breeds contempt. In each of his reports it will be found that he never failed to give credit to those who had assisted him in their preparation.

"He believed the engineering profession to be one of the greatest, and maintained an ethical standard such as few can hope to measure up to. By the life and conduct of such men as he, the engineering profession will be raised to its highest plane, and the moral tone of all with whom they come into contact cannot fail to be enriched. With all who had the privilege of working with Mr. Adams, and with all who came into contact with him in any walk of life, his memory will live forever as that of a true gentleman and an eminent—or rather a pre-eminent—engineer."

Mr. Adams was married on December 18th, 1889, to Mary Gemmell, of Topeka, Kans., who, with two sons and three daughters, survives him. Robert, the eldest, is a student in the Engineering Department of Stanford University, and two of the daughters are students in the University of California.

Mr. Adams died at his home in Oakland, Cal., on September 17th, 1913, of pneumonia, after an illness of less than one week. In his death the Engineering Profession has lost one of its great leaders; the community an active and efficient instrument for civic and social betterment; and his family and friends a wise counselor and a genial, lovable companion.

Mr. Adams was elected a member of the American Society of Civil Engineers on October 2d, 1895. From 1907 to 1909 he served as a

Director. He was President of the San Francisco Association of Members of the American Society of Civil Engineers during 1907. He was a member of the Franklin Institute, the American Academy of Political and Social Science, the Technical Society of the Pacific Coast, the Pacific Association of Consulting Engineers, and of the First Presbyterian Church of Oakland. He held important offices in his church, and was Vice-President and Director of the Young Men's Christian Association of Oakland. He was also a member of the Delta Tau Delta and Sigma Xi college fraternities.

RICARDO MANUEL ARANGO, M. Am. Soc. C. E.*

Died January 24th, 1914.

Ricardo Manuel Arango, Consulting Engineer for the Republic of Panama, died at his home in the City of Panama, on January 24th, 1914, after a long illness. He was born on December 25th, 1864, and was graduated in 1887, with the degree of C. E., from the Rensselaer Polytechnic Institute, at Troy, N. Y. His first work after graduation was a survey, under the direction of the late Pedro J. Sosa, M. Am. Soc. C. E., a prominent engineer of Panama, of 500 000 hectares of land in the Province of Bocas del Toro, conceded by the Colombian Government to the French Canal Company. This party did the first surveying work in that region.

In 1889, Mr. Arango was appointed by the Colombian Government to take charge of surveys made necessary by a controversy between it and the Panama Railroad, over the filling of the present site of Colon. In 1890, he was engaged on the railroad which was built to haul to the coast the products of the manganese mines of Viento Frio, Province of Colon. He assisted in the surveys and plans for providing the City of Panama with water from the Juan Diaz River, but this development was never realized.

Mr. Arango took an active part in the Revolution of 1903 by which Panama secured its independence from Colombia, being of great assistance to his father, José A. Arango, who was a member of the original Junta of Separation. As member of the Municipal Council of the City of Panama, Mr. Arango signed the Act of Independence, and was appointed the first Chief Engineer of the new Republic.

Under John F. Wallace, Past-President, Am. Soc. C. E., Chief Engineer of the Isthmian Canal Commission, Mr. Arango was Consulting Engineer for sanitary work in the City of Panama. Later, he was made Division Engineer of the Bureau of Meteorology and

* Memoir prepared by Alex. P. Crary, Assoc. M. Am. Soc. C. E.

River Hydraulics. As head of this Bureau, he installed the first seismograph on the Isthmus, and established the gauging station at Alhajuela, on the Chagres River, from which advices of any floods could be telophoned in advance to those in charge of work on the lower reaches of the river. His services with the Isthmian Canal Commission were terminated in the fall of 1908. Soon afterward Mr. Arango was appointed Minister Plenipotentiary and Envoy Extraordinary for the Republic of Panama to the Court of St. James, which post he was compelled to relinquish in 1909 on account of ill health. In 1910, he was again made Chief Engineer of the Republic, and, later, Consulting Engineer, which position he held at the time of his death.

He was a Member of the Instituto de Ingenieros de Chile, the Seismological Society of America, and the Sociedad de Ingenieros, Arquitectos y Agrimensores de Panama, of which latter he was the founder. On September 21st, 1899, he was married to Miss Maria Lewis, who, with five children, survives him.

Mr. Arango was a man who loved his profession. Even during his long illness, when he had lost almost completely the use of his hands, he took great delight in reading and studying engineering. In a wonderful manner he had trained his mind so that he could transform equations, construct graphical diagrams, and do things mentally which are generally accomplished by paper and pencil. He was always genial and willing to help and give advice. He stood up courageously under his sickness though it deprived him of the many pleasures and much work which make life worth living. He was always interested in public affairs, and, as long as he could, he took an active part in them. In his death, Panama loses one of its best and most progressive citizens.

Mr. Arango was elected an Associate Member of the American Society of Civil Engineers on September 2d, 1896, and a Member on February 6th, 1906.

JOHN WILLIS HAYS, M. Am. Soc. C. E.*

DIED DECEMBER 14TH, 1913.

John Willis Hays was born at Oxford, N. C., on March 14th, 1861, and was christened with the name borne by his father and his grandfather. He was educated at local private schools and the University of North Carolina.

As a young man Mr. Hays did engineering work under the late Benjamin D. Frost, M. Am. Soc. C. E., of Massachusetts, who had distinguished himself in connection with the building of the Hoosac Tunnel in that State, and was, at the time in question, engaged in laying out a route for the Oxford and Henderson Railroad. Later,

* Memoir prepared by Francis B. Hays, Esq.

Mr. Hays joined the field staff of William C. Kerr, State Geologist of North Carolina, where he acquired experience and developed ability which gained for him an appointment as a Topographer in the United States Geological Survey. This position he held for many years, first in the Appalachian Mountains, and later in the Rockies, during the summer, and at Washington in winter.

In addition to the remarkable thoroughness of his engineering work, Mr. Hays was distinguished for the unusual excellence of his drafting. Indeed, his artistic ability was marked, and had he chosen Art as a vocation, his success would doubtless have been gratifying. Nor was his facility with the pen confined to drafting and free-hand sketching, for many of his exciting mountain adventures were reduced to manuscript and published in such periodicals as *The Outing Magazine, The Youth's Companion,* and the newspapers of Washington and other cities. He was also an occasional contributor to journals devoted to engineering matters.

Leaving the Government service in the early Nineties for work in which his individuality would have freer play, Mr. Hays became City Engineer of Petersburg, Va., having a few years previously married the daughter of one of the leading citizens of that place, who, with a large family, survives him. While in that position he laid out Ferndale Park, constructed the high-service reservoir, and in other ways left his impress on the city's physical characteristics.

Mr. Hays' nature was such that he could not be contented in a political position. He knew of but one course to pursue, and that was the one of absolute rectitude as he saw it, and he was remarkably free from moral strabismus. Consequently, he left the public position to engage in private work. Although frequently urged to return to the City Engineership at an increased salary, he steadfastly and politely declined.

In the meantime, he built up a private practice which became more remunerative as the years went by, and he was able to command the fee of an expert for a mere opinion. Shortly before his death, he had completed the development of the Walnut Hills Section, reached from the City of Petersburg proper by an iron viaduct over a broad gorge which latter had long rendered it practically inaccessible. He developed the water power at Roanoke Rapids, N. C., and did municipal work at Mt. Airy, Dunn, Kingston, and other towns in his native State, as well as at Emporia, Blackstone, and many other points in the State of his adoption.

Physically, Mr. Hays was a man of magnificent mould, having been considerably more than 6 ft. in height, erect and clean-limbed, and carrying his two hundred and odd pounds without the suggestion of a surplus ounce of flesh. He was cut down at the zenith of his powers by thrombosis induced by a fall sustained while engaged in field work.

Mr. Hays was Master of Blandford Lodge of Masons, President of the Petersburg Benevolent Mechanics' Association, an Elk, and, for several years, had conducted with marked success a men's Sunday-school class. He was a man of vigorous mind, keen intellect, catholic tastes, strong personality, a faithful friend, and a true gentleman.

Mr. Hays was elected a Member of the American Society of Civil Engineers on June 5th, 1901.

LUTHER REESE ZOLLINGER, M. Am. Soc. C. E.*

Died October 21st, 1913.

Luther Reese Zollinger, the fifth son of William George and Susannah (Spece) Zollinger, was born on April 25th, 1865, in Harrisburg, Pa. He received his early education in the public schools of his native city, having been graduated from the High School in 1883. Of his early life his most intimate boyhood acquaintance, Dr. C. R. Phillips, of Harrisburg, writes:

"No member of our class in High School, where we were associated for four years, held his classmates by the mere strength of a lovable personality as did Luther Reese Zollinger. We were all devoted to him because of that indescribable something he had, which, unfortunately, is uncommon, and made us certain of his sincerity of purpose and life. He was honor man in our class in High School, easily so too, and I have always felt that he was honor man in our class at Lehigh, though it is true that one or two others had a few per cent. higher standing, according to the poor, insufficient ratings which, as things educational stand, must still be taken to measure a man's mind and ability. Of all the men whom I have known, he had, I always felt, the best mind for mathematical work. In saying this I, of course, except a few neurotics whose mathematical ability always is a part of an unstable mental equilibrium."

In 1884, Mr. Zollinger entered Lehigh University, and was graduated in 1888 with the degree of Civil Engineer. He was President of his class in the Sophomore year, Editor of the College *Epitome,* and Business Manager and Editor of the *Engineering Journal,* and had a Commencement appointment. During his 4 years in college he never ranked lower than fifth, and for the last 3 years he ranked second in a class varying from 101 to 66 men.

Mr. Zollinger's entire professional career was spent in the service of the Pennsylvania Railroad Company, in which he rose by his own efforts and merit from the humble post of Chainman to the position of trust and responsibility which he occupied at the time of his death. Starting in the office of the Assistant Engineer, Middle Division, at Harrisburg, Pa., on March 11th, 1889, he received his first professional

* Memoir prepared by J. F. Murray and C. J. Parker, Members, Am. Soc. C. E.

experience at the time of the great floods throughout Pennsylvania, in May and June of that year, which proved so destructive to the railroads and other internal improvements in various parts of the State, especially in the vicinity of Johnstown, and along the Susquehanna, Juniata, and Connemaugh Rivers.

He held successively, and for varying periods from 1889 to 1905, the various positions of Transitman in the office of the Engineer of Maintenance of Way, Pennsylvania Railroad Division, at Altoona; Assistant to Assistant Engineer, Philadelphia Division, at West Philadelphia; Assistant Supervisor, Division No. 8, Middle Division, Pennsylvania Railroad, at Spruce Creek; Supervisor, Division No. 28, at Norristown; Supervisor, Division No. 1, Philadelphia Division, at West Philadelphia; Assistant to Principal Assistant Engineer, and Principal Assistant Engineer, Pennsylvania Railroad Division, at Altoona.

On April 1st, 1905, Mr. Zollinger was promoted to the position of Engineer of Maintenance of Way, General Office, Philadelphia, which position he held at the time of his death. In this capacity he was in charge and had direct control of the Maintenance of Way Department, in so far as was necessary to insure the efficiency of the Department and adherence to the standards of the Company. He also had charge of the preparation of all maintenance of way plans, the issuing of instructions in regard to adherence to the same, and personally examined all bridges and other structures, reporting on their condition and making recommendations in connection therewith, as well as with respect to other matters relating to maintenance of way.

In addition to these duties, Mr. Zollinger, in 1905-07, was in charge of the construction of extensive yards and yard facilities at Morrisville on the New York Division, and at Shire Oaks on the Monongahela Division; he was also appointed as Chairman of various committees, by the General Manager, to make experiments, investigations, and reports on sundry matters relative to the economical construction and maintenance of track. One of these reports,* dated March, 1911, covers extended experiments to determine the necessary depth of stone ballast. Of this report, W. C. Cushing, M. Am. Soc. C. E., Chief Engineer, Maintenance of Way, Pennsylvania Lines West, says: "These tests are the most extensive of the kind ever conducted in this country, and will be found of great interest and value to railway engineers."

Mr. Zollinger traveled extensively, both in this country and in Europe, and his wonderful powers of observation, retentive memory, quick comprehension, enthusiasm, and capacity for work, made him invaluable to the company he served. As an example of his enthusiasm and wonderful resourcefulness, he purchased a tract of land at

* *Proceedings*, American Railway Association, Vol. 13.

Merion, Pa., on which was an old, abandoned stable, and with the aid of his architect transformed it into a beautiful home of the English manor type,* utilizing for the interior finish and decoration the materials from an old mansion formerly on the estate.

His personality was peculiarly lovable and attractive, and left its impress deep on all who met him. One felt immediately that he was a man worth knowing, an impression that association only strengthened and confirmed. Big of body, handsome of feature, with great magnetism, his winning personality drew all classes of men to him, and he held them with hooks of steel by his genial, broad-minded, and tolerant disposition. He was by nature a man of essentially social tendencies. Of fine intellectual gifts, broad education, wide acquaintance with the best literature, discriminating taste in the fine arts, varied experience with many classes of men, an observant traveler, with a retentive memory, great fund of anecdote and reminiscence, and a keen sense of humor, he was always entertaining in conversation. These qualities, with his loyalty, kindness, generosity, and buoyance of spirits, made him a most interesting and delightful companion.

Full of sympathy, he was never too busy to give time to the troubles of his friends or counsel to the youth seeking opportunity. It is not often that a man who has attained high professional standing shows a live interest in the young man just beginning his life work, but Mr. Zollinger was the champion of youth, with an unbounded faith in its capabilities and the fulfilment of its aspirations. This faith he exemplified in a very striking and practical way, for he took more than one young man into his home, gave them his friendship and confidence, inspired them by his manliness, educated them at his own expense, and launched them on honorable and useful careers in life. It was a great satisfaction and source of much pride to him to know that each of these young men fully justified the trust placed in him.

His genial nature, overflowing with good fellowship, so attractive to his many friends, was carried into his domestic circle and was one of the finest phases of his life, as seen by his intimate friends. He never married, and his devotion to his sister can only be described as beautiful. Nothing he did for her was ever felt to be a sacrifice. His happiness was only reached through her happiness. Such affection was warmly reciprocated, and his home was made to him a place of charm as well as of rest.

Mr. Zollinger's dominant characteristic was courageous and vigorous manhood. His power of mind and body was shown in his capacity for concentration and the accomplishment of work. He was of quick discernment and swift comprehension, and his strength made him positive in his convictions and strenuous in maintaining them. With

* Described by the architect in *House and Garden*, December, 1912.

his strong character and self-respect, he could never play the courtier, and he detested to receive the flattery he would never apply to others, not even in the form of studied deference which, at times, might have gained him personal advantage.

Enthusiasm in regard to whatever matter he took up, either work or recreation, was another feature of his character, and made him a natural leader. He was a lover of the beautiful in Nature and art. Nature in all her forms appealed strongly to him. Some of his happiest hours were spent in roaming among the trees of the forest or the flowers of his own garden, whose characteristics and history he knew so well. He kept in touch with the advancement in science and all questions of the day, and could enjoy intelligently and appreciate intercourse with men versed in many forms of learning.

Men of great professional attainments and mental power are often deficient in those qualities of the heart which make them loved as well as honored, but these qualities Mr. Zollinger possessed in an unusual degree. He had intellectual superiority, and he was a man of honorable achievement, but those who knew him well think of him and love him for his broad-gauged, liberal-minded, vigorous manhood and spotless integrity. He was chivalric in thought and deed, and the courteous gentleman always. Honor, truth, and duty were the stars by which he steered his course, and in him was embodied the highest type of manhood. Of splendid physique, clean and wholesome morals, a fascinating personality with a highly cultivated mind, he died in the prime of brilliant achievements. The exquisite charm of his friendship can be known only to those who enjoyed it; to them it was a benediction, and his death an irreparable loss. In a generation we shall not see his like again.

Mr. Zollinger had complained of ill health for several months prior to his death, though he did not relinquish any of his work, and had just returned from the annual track inspection when he was stricken with apoplexy, on the morning of October 16th, and died at his home in Merion, Pa., on October 21st, 1913.

He was a Member of the American Railway Engineering Association and the Delta Upsilon Fraternity.

Mr. Zollinger was elected a Member of the American Society of Civil Engineers on March 6th, 1901.

MURRAY FORBES, Assoc. M. Am. Soc. C. E.*

Died December 28th, 1913.

Murray Forbes was born in Philadelphia, Pa., on June 23d, 1863. His father, Dr. William S. Forbes, was Professor of Surgery at Jef-

* Memoir prepared by W. C. Hawley and G. W. Hutchinson, Members, M. Am. Soc. C. E.

ferson Medical College. His mother Celinere (Sims) Forbes, was a sister of J. C. Sims, Secretary of the Pennsylvania Railroad Company, and Judge Clifford Sims of the Superior Court of New Jersey.

Mr. Forbes received his early education at Rugby Academy and at Dr. Farries' School, in Philadelphia, and entered the Art School of the University of Pennsylvania at the age of fourteen. When he was 17, he entered the Pennsylvania Railroad shops at Altoona, Pa., where he took the regular 4 years' apprenticeship course. He was then employed in the shops for some months, being afterward transferred to Derry, Pa., where for 4 years he was Assistant Road Foreman of Engines on the Pittsburgh Division of the Pennsylvania Railroad.

He resigned this position in 1888 and went to Greensburg, Pa., where he took charge of the construction of the plant of the Westmoreland Water Company. Mr. Forbes continued as the executive head of this Corporation until his death. Under his direction, the plant was extended and, with allied corporations, served the communities along the Main Line of the Pennsylvania Railroad west from Greensburg as far as Irwin, and south along the Southwest Branch as far as Youngwood. A total of 50 000 people are supplied, in addition to large industrial and mining plants, by the companies in which he was the guiding spirit, and this, too, in a section where, because of the broken topography and the dumping of mine drainage into the streams, the operating conditions are exceedingly difficult and expensive. Mr. Forbes was in responsible charge of the design and construction of the plant, and, at the time of his death, he was, and had been for many years, Manager, Secretary, and Treasurer of the Company and its allied interests.

Mr. Forbes' success at Greensburg resulted in his being chosen to take charge of the construction and operation of water plants for the Derry Water Company at Derry, Pa., and the Dennison Water Supply Company at Dennison, Ohio, serving as Manager, Secretary, and Treasurer of both these corporations. He also had a large financial interest in all three companies.

Mr. Forbes was recognized as an authority in matters pertaining to water-works construction, operation, and valuation. He was employed in a number of water-works cases, either as an expert witness or as a member of boards of arbitration. He assisted in organizing the Pennsylvania Water Works Association and served for 3 years as its President. He was also a Member of the American Water Works Association, the New England Water Works Association, the Engineers' Society of Western Pennsylvania, the Engineers' Society of Pennsylvania, and the Union League Club of Philadelphia. He was a Mason and a member of the Protestant Episcopal Church.

Mr. Forbes was a delightful character, always kind and courteous, a man who made friends readily and who kept them. Of strong convictions, he was also a diplomat. Although for many years he had acted as Manager of the Water Company which served the town in which he lived—and under difficult conditions and with necessarily high rates—he was popular and highly respected in that community. An editorial in a Greensburg newspaper, at the time of his death, reads, in part, as follows:

"Mr. Forbes was in many ways superior. He was a factor in the affairs of his town and of his county. He was intensely human. He was a master of his profession, and he controlled because he knew. A useful man has been taken from this community."

In 1893, Mr. Forbes was married to Miss Ethel Parvin, of Philadelphia, who, with five children, survives him.

Mr. Forbes was elected an Associate Member of the American Society of Civil Engineers, on June 5th, 1907.

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"THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS." J. A. L. WADDELL. (To be presented April 15th, 1914.)

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William P. Morse

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OF THE

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OF

CIVIL ENGINEERS

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(INSTITUTED 1852)

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Edited by the Secretary, under the direction of the Committee on Publications.

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

March 18th, 1914.—The meeting was called to order at 8.30 P. M.; T. Kennard Thomson, Director, in the chair; Chas. Warren Hunt, Secretary; and present, also, 106 members and 9 guests.

A paper by C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie and C. O. Carey, entitled "Report on a Series of Tests on Concrete Columns Reinforced with a Spiral of Steel", was presented by the Secretary, who also read a communication on the subject from Edward Godfrey, M. Am. Soc. C. E. The paper was discussed orally by A. W. Buel, M. Am. Soc. C. E.

The Secretary announced the following deaths:

WILLIAM AUGUST HUNICKE, of St. Louis, Mo., elected Associate Member, March 2d, 1904; Member, January 7th, 1913; died March 9th, 1914.

GEORGE WESTINGHOUSE, of Pittsburgh, Pa., elected Member, January 7th, 1891; died March 12th, 1914.

Adjourned.

April 1st, 1914.—The meeting was called to order at 8.30 P. M.; Vice-President Gardner S. Williams in the chair; Charles Warren Hunt, Secretary; and present, also, 121 members and 14 guests.

The minutes of the meetings of February 18th and of March 4th and 11th, 1914, were approved as printed in *Proceedings* for March, 1914.

A paper by F. Lavis, M. Am. Soc. C. E., entitled "The Gauge of Railways, with Particular Reference to Those of Southern South America," was presented by the author who illustrated his remarks with lantern slides. The paper was discussed orally by Philip W. Henry, M. Am. Soc. C. E.

The Secretary announced the election of the following candidates on April 1st, 1914:

As Members

Alexander Allaire, Vancouver, B. C., Canada
Jacob Thompson Bullen, Shreveport, La.
Lee Llewellyn, Pittsburgh, Pa.
Charles Harvey MacCulloch, Schenectady, N. Y.
Henry Matson Waite, Cincinnati, Ohio

As Associate Members

Francis Neal Baldwin, Alexandria, La.
Charles Dana Sayres Clarkson, Washington, D. C.
Arthur Mandeville Compton, Davenport, Iowa
Dexter Parshall Cooper, New York City
Clare Harmon Currier, Webster City, Iowa
Willis Johnson Dean, San Diego, Cal.
Carlton Robb DeGraff, Amsterdam, N. Y.
Charles Edson Douglas, Meadville, Pa.
Howard Howell George, Newark, N. J.
David Herrick Goodwillie, Toledo, Ohio
Ross Elroy Hamilton, Coshocton, Ohio
William Frederick Hicks, Lethbridge, Alberta, Canada
William Fleming Holliday, Palmerton, Pa.
Augustus Crane Hone, New York City
José Petronio Katigbak, Manila, Philippine Islands
Michael Nikanorovitch Lebedeff, Denver, Colo.
Charles Wells Linsley, Oswego, N. Y.
David Ludwig Loewe, Berlin, Germany
David Cadenhead MacDougall, Westfield, N. J.
Patrick James Malley, Boston, Mass.
Hamilton Vincent Miles, Manila, Philippine Islands
Robert Orrell Morrison, Monroe, La.
William Henry Nalder, Great Falls, Mont.
Charles Frederick Puff, Jr., Philadelphia, Pa.

Martin John Reinhart, Oklahoma City, Okla.
Karl DeWitt Schwendener, Los Angeles, Cal.
Clarence Edmund Seage, New York City
Samuel Alan Sloan, Philadelphia, Pa.
Alexander Linn Trout, Detroit, Mich.
Howard Moore Turner, Turners Falls, Mass.
James Hubert van Wagenen, Washington, D. C.
Rodman Wiley, Frankfort, Ky.
George John Woehrlin, Brooklyn, N. Y.
Benjamin Russell Wood, Manila, Philippine Islands
Berthold Wuth, Oakland, Cal.

As Juniors

Edgar Dow Gilman, Madison, Wis.
Arthur Francis Holland, New Martinsville, W. Va.
Walter Edgar Jessup, South Pasadena, Cal.
Harvey Stone Johnson, Utica, N. Y.
John Jacob Krauss, Ann Arbor, Mich.
Frederic Oganam Xavier McLoughlin, New York City
Edward Anders Malmquist, Astoria, N. Y.
Victor Mayper, New York City
George John Meise, New York City.
Emile Leonard Rimbault, Jr., New York City
Leon Emerson Swartz, Altoona, Pa.
Charles Randolph Thomas, 3d, State College, Pa.
George Sparkman Ward, Boca Grande, Fla.
John Thad Whitney, Wheeling, W. Va.

The Secretary announced the transfer of the following candidates on April 1st, 1914:

From Associate Member to Member

Gustav Storm Bergendahl, Chicago, Ill.
Ralph Peters Black, Charleston, W. Va.
Charles Bernard Buerger, New York City
La Vern John Charles, Elephant Butte, N. Mex.
Washington Irving Lex, Philadelphia, Pa.
Samuel Jacob Ott, Rutherford, N. J.
Joshua Fielden Ramsbotham, Melbourne, Victoria, Australia
Frederick Howard Tillinghast, Lahontan, Nev.

From Junior to Associate Member

John Nixon Brooks, Trenton, N. J.
Nathan Booker Buchanan, Tupelo, Miss.
Lester Lyman Coleman, Maricopa, Cal.

Torris Eide, New York City
George Rodman Goethals, Culebra, Canal Zone, Panama
Harry Milton Lynde, West Raleigh, N. C.
Kenneth Dunham Owen, Montclair, N. J.
Ralph Edgar Spaulding, Suffield, Conn.
Newton Benjamin Wade, Millville, N. J.

The Secretary announced the following deaths:

William Dunbar Jenkins, of Natchez, Miss., elected Member. September 7th, 1887; died March 12th, 1914.

Benjamin Franklin Morse, of East Cleveland, Ohio, elected Member, July 12th, 1877; died February 24th, 1914.

Roger Tifft Holloway, of New York City, elected Junior, May 31st, 1910; Associate Member, May 7th, 1913; died March 13th, 1914.

Adjourned.

SPECIAL MEETINGS

April 2d, 1914.—The meeting was called to order at 2.30 P. M.; Vice-President Gardner S. Williams in the chair; Chas. Warren Hunt, Secretary; and present, also, 51 members and 8 guests.

The Chairman announced that the meeting had been called for the purpose of discussing the Progress Report of the Special Committee on the Valuation of Public Utilities.

The discussion was opened by C. Tombo, Assoc. M. Am. Soc. C. E., who read a communication on the subject from Charles Hansel, M. Am. Soc. C. E. The report was discussed also by Messrs. George F. Swain, D. W. Lum, J. H. H. Muirhead, J. N. Dodd, C. R. Harte, S. Whinery, J. M. Schreiber, and M. L. Byers.

On motion, duly seconded, the meeting was adjourned until 8.30 P. M.

April 2d, 1914.—The meeting was called to order at 8.30 P. M.; Vice-President Williams in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 41 members and 4 guests.

The Assistant Secretary read a communication on the subject from H. M. Stone, M. Am. Soc. C. E., and the discussion was continued by Messrs. J. H. H. Muirhead, T. Kennard Thomson, J. P. Snow, George F. Swain, Gardner S. Williams, and Frederic P. Stearns.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

March 4th, 1914.—The Board met at 3.15 P. M.; President McDonald in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Edwards, Endicott, Fuller, Gerber, Hodge, Keefer, Leonard, Ockerson, Smith, Swain, Thomson, Tuttle, and Williams.

The Committee appointed to consider the question of the payment of mileage to Directors attending Annual Conventions reported that in its opinion the payment of such mileage was not justified by the conditions, nor in the best interests of the Society, and on motion, duly seconded, the report was received and its recommendations adopted as the action of the Board.

The following Resolutions were adopted:

"*Resolved:* That it is the sense of this meeting of the Board of Direction that it would be wise to enlarge the work of the Society by the encouragement of the work of its Special Committees, including research work, and by the appropriation of some of its surplus funds for this purpose."

"*Resolved:* That the Finance Committee be requested to report to the Board the amount which, in its opinion, it would be proper to appropriate for such purpose during 1914."

Letters of protest against the issue of an unsigned postal card relating to the Amendments to the Constitution were presented, and the matter was referred to the Publication Committee for report.

The Constitution of the Spokane Association of Members of the American Society of Civil Engineers was approved.

The resignation of Frank M. Kerr, M. Am. Soc. C. E., as Chairman of the Special Committee on Floods and Flood Prevention, was received and accepted. C. McD. Townsend, M. Am. Soc. C. E., was appointed Chairman of that Committee to fill the vacancy.

Ballots for membership were canvassed, resulting in the election of 6 Members, 20 Associate Members, and 14 Juniors, and the transfer of 7 Juniors to the grade of Associate Member.

Five Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

April 1st, 1914.—The Board met at 3.35 P. M.; Vice-President Williams in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Edwards, Endicott, Fuller, Gerber, Haskell, Hodge, Keefer, Leonard, Swain, Thomson, and Tuttle.

William Douglas Pickett, M. Am. Soc. C. E., was elected an Honorary Member by the unanimous vote of the entire Board of Direction and all living Past-Presidents.

Upon the recommendation of the Publication Committee the following was unanimously adopted as the action of the Board:

"That the unofficial use of the Society's Mailing List be restricted to the membership of the Society for the transmission of information on Society matters in communications bearing the name of the member or members responsible therefor."

"That in view of certain criticisms of the 'mode of procedure' heretofore prescribed by the Board in the appointment of the Nominating Committee (Article VII, Sec. 2 of the Constitution), and as to the form used in preparing certain letter-ballots, the following resolutions be adopted:

"*Resolved:* That in the matter of the submission of suggestions for the Nominating Committee, a date for closing the receipt of both the first and second set of suggestions be fixed each year by the Board of Direction, and that all such suggestions be opened and canvassed by the Board.

"*Resolved:* That the provisions of Article VII, Sec. 6, of the Constitution, which relate to letter-ballots for election of officers, shall in future apply to all letter-ballots on other matters submitted to the vote of the Corporate Members of the Society."

The Publication Committee reported that it had spent more than six months in investigating the cost of printing and in securing Competitive Bids, both from printers in New York City and out of town, and recommended that a new contract be entered into with the Evening Post Job Printing Office (which is now doing the work), said contract to run until August, 1915.

Upon the recommendation of the Publication Committee, a circular was ordered, to ascertain the views of the entire membership as to the use of thin paper in our publications.

A Code of Ethics, presented some time ago by a Committee composed of Messrs. Endicott, Wallace, and Hodge, was considered and revised, and the following was ordered sent out to the membership of the Society, and to be brought up at the Business Meeting of the Annual Convention with the recommendation of the Board that it be adopted by the Society:

Code of Ethics

"It shall be considered unprofessional and inconsistent with honorable and dignified bearing for any member of the American Society of Civil Engineers:

"1. To act for his clients in professional matters otherwise than as a faithful agent or trustee, or to accept any remuneration other than his stated charges for services rendered his clients.

"2. To attempt to injure falsely or maliciously, directly or indirectly, the professional reputations, prospects, or business, of another Engineer.

"3. To attempt to supplant another Engineer after definite steps have been taken toward his employment.

"4. To compete with another Engineer for employment on the basis of professional charges, by reducing his usual charges and in this manner attempting to underbid after being informed of the charges named by another.

"5. To review the work of another Engineer for the same client, except with the knowledge or consent of such Engineer, or unless the connection of such Engineer with the work has been terminated.

"6. To advertise in self-laudatory language, or in any other manner derogatory to the dignity of the profession."

A report was received from a Committee consisting of Messrs. Bates, Bensel, Endicott, Ockerson, and Swain, relating to the payment of mileage and to other expenses of Special Committees, which report, after slight modification, was approved and adopted as the action of the Board. As adopted, the action reads as follows:

"The Board of Direction of the American Society of Civil Engineers having considered the advisability of allowing mileage to members of Special Committees, and also the advisability of allotting funds to such Committees to cover the necessary general expenses incident to the work assigned them, has adopted the following:

"(1) Since one of the declared objects of the Society is 'the advancement of engineering knowledge and practice', it is recommended that at the beginning of each calendar year a budget be prepared and adopted by the Board of Direction covering the various classes of expenditures, including a fund for research or Special Committee work, of such amount as in the opinion of the Board can properly be diverted to such purposes.

"(2) That when a Special Committee is appointed it shall at once submit to the Board of Direction a project outlining the scope and character of the work proposed, together with an estimate of the cost thereof.

"(3) On the receipt of such project covering the nature of the work proposed and the estimated cost thereof, the Board of Direction may alter or amend such project and define the general lines along which the work should be conducted.

"(4) When a project has been approved by the Board of Direction the said Board may allot from the surplus funds of the Society such amount as in its opinion may be necessary and proper for the work proposed.

"(5) Any part of the allotment made for the use of a Special Committee may be used for the payment of personal expenses to the extent of mileage at a rate of three cents per mile, when transportation is actually paid by members of the Committee.

"(6) Satisfactory vouchers covering the expenditures in detail must be submitted to the Board, and must be approved by the Committee on Finance before being paid; provided, however, that the Board of Direction may advance to such Committees such sums as may, in its opinion, be deemed necessary, subject, however, to such final settlement and approval as herein provided.

"(7) It is further recommended that funds from the allotments made by the Board be subject to draft by the Chairmen of the respective Committees, from time to time, as needed, when accompanied by a statement satisfactory to the Board of Direction as to the proposed application of the funds called for."

Ballots for membership were canvassed, resulting in the election of 5 Members, 35 Associate Members, 14 Juniors, and the transfer of 9 Juniors to the grade of Associate Member.

Eight Associate Members were transferred to the grade of Member.

Applications were considered, and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

May 6th, 1914.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion as follows: "The Determination of Safe Yield of Underground Reservoirs of the Closed-Basin Type", by Charles H. Lee, Assoc. M. Am. Soc. C. E.; and "Cinder Concrete Floors", by Guy B. Waite, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

May 20th, 1914.—8.30 P. M.—At this meeting, three papers will be presented for discussion as follows: "California Practice in Highway Construction", by W. C. Hammatt, M. Am. Soc. C. E.; "Reinforced Concrete Docks: Foreign and American Structures. Failures, Costs, and General Considerations", by Harrison S. Taft, Esq.; and "Huacal Dam, Sonora, Mexico", by H. Hawgood, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-sixth Annual Convention of the Society will be held at Baltimore, Md., from June 2d to 5th, 1914, inclusive.

Arrangements are in charge of Special Committees, and the programme will be issued very soon in a circular to the membership. Although the details cannot now be given, it is expected that, in addition to the Business and Professional Meetings, there will be all-day Excursions to Annapolis and to Sparrows Point, also to engineering works in the vicinity of Baltimore, as well as a Golf Tournament at the Country Club, and a Reception and Dance, to be given by the Engineers' Culb of Baltimore.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work, the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet,

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., and, until further notice, will take place at the lunch room of the Denver Dry Goods Company.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

March 14th, 1914.—The meeting was called to order; President Ridgway in the chair; Roger W. Toll, Secretary; and present, also, 30 members and guests, including ladies.

The minutes of the meeting of February 14th, 1914, were read and approved.

A paper on "The Building and Shop Erection of the Sluice and Penstock Gates for the Elephant Butte Dam in New Mexico", was presented by L. R. Hinman, Assoc. M. Am. Soc. C. E., who illustrated his remarks with stereopticon views and with motion pictures taken at the Dam. The subject was discussed by Messrs. George F. Roehrig, Jr., and H. S. Crocker.

A vote of thanks was tendered Mr. Hinman for his interesting paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club.

At the meeting of the Association on December 29th, 1913, the new Chairman, John Ruddle, M. Am. Soc. C. E., was installed, and Messrs. Park A. Dallis and G. R. Solomon were appointed members of the Executive Committee. T. P. Branch, Assoc. M. Am. Soc. C. E., was elected Secretary.

Philadelphia Association

On December 22d, 1913, the Philadelphia Association of Members of the American Society of Civil Engineers was organized with the following officers: George S. Webster, President; Richard L. Humphrey and F. Herbert Snow, Vice-Presidents; John Sterling Deans, J. W. Ledoux, Edgar Marburg, and H. S. Smith, Directors; S. M. Swaab, Treasurer; and W. L. Stevenson, Secretary. The meetings of the Association will be held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

(Abstract of Minutes of Meeting)

February 28th, 1914.—The meeting was called to order; President Hopson in the chair; Charles J. McGonigle, Secretary; and present, also, 17 members and guests.

The minutes of the preceding meeting were read and approved.

Communications from the Executive Secretary of the International Engineering Congress, 1915, and from the San Francisco Association of Members of the American Society of Civil Engineers, were read.

W. H. Daly, Commissioner of Public Utilities, and D. D. Clarke, M. Am. Soc. C. E., Engineer of the Water Board, addressed the meeting in support of the installation of water meters in the City of Portland, and the subject was discussed by Messrs. Hopson, Graves, Henny, Taylor, Stubblefield, and Randlett.

A motion was adopted unanimously endorsing the position taken by Commissioner Daly in the matter of the installation of meters.

A vote of thanks was tendered Messrs. Daly and Clarke by the Association.

Adjourned.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year: Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

Southern California Association

On January 5th, 1914, the Southern California Association of Members of the American Society of Civil Engineers held its first meeting at the University Club, Los Angeles, Cal., and elected the following officers: J. B. Lippincott, President; Charles T. Leeds, First Vice-President; George S. Binckley, Second Vice-President; W. K. Barnard, Secretary; and Charles H. Lee, Treasurer.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, C. S. MacCalla; Vice-President, U. B. Hough; Second Vice-President, Morton Macartney; Secretary-Treasurer, A. D. Butler.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, Room 6, City Bank and Trust Company Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From March 3d to April 1st, 1914)

DONATIONS*

INDUSTRIAL CHEMISTRY FOR ENGINEERING STUDENTS.

By Henry K. Benson. Cloth, 7¾ x 5 in., illus., 14 + 431 pp. New York, The Macmillan Company, 1913. $1.90.

The author's aim in this book, it is stated, has been to describe from the chemical standpoint, the more common materials used in engineering, particular emphasis being laid on the occurrence, methods of manufacture, the properties, and, to a limited extent, the uses of such materials. The subject-matter has been compiled and elaborated, it is said, from lecture notes used by the author during the last eight years in a course of industrial chemistry for second-year engineering students, and each chapter has been made sufficiently complete in itself so that the various topics may be taken up and studied in any order preferred by the teacher. The author presupposes a knowledge of elementary physics and general chemistry on the part of the reader, and it is hoped that the book will give the student a working knowledge of the chemistry of materials and processes with which he will have to deal as well as the ability to interpret chemical analyses and to apply them in the preparation of specifications and in experimental research work. Such topics as fuels and combustion, clay products and cement, which are of the greatest interest and importance to engineers, are discussed quite fully, it is stated, and bibliographies of the subject discussed are given at the end of each chapter. These bibliographies have been carefully compiled and brought up to date, the arrangement being alphabetical by authors in chronological order, and it is hoped that they will be helpful to practising chemists and engineers, as well as to students. The Contents are: General Processes and Apparatus; The Atmosphere; Industrial Water; Combustion and Destructive Distillation; Solid Fuels; Liquid and Gaseous Fuels; Petroleum and Lubricating Oils; The Manufacture of Pig Iron; Commercial Forms of Iron and Steel; The Industrial Alloys; Clay Products; Hydraulic Cements and Lime Products; Paving Materials and Wood Preservation; Paint and Varnish Materials; Plastics for Electrical Insulation; Cellulose Products; Explosive Materials; Index.

THE BUREAU OF SUPPLIES

Of the Department of Water Supply, Gas and Electricity, New York City: A Report to Hon. Henry S. Thompson, Commissioner. By Elihu Cunyngham Church, Secretary of the Department and Chief of the Bureau of Supplies. Cloth, 10¼ x 7 in., illus., 93 pp. New York, M. B. Brown Printing & Binding Co., 1913. (Donated by the Author.)

In his letter of transmittal, the author states that more than two years ago he was directed to take charge of the Bureau of Supplies of the Department of Water Supply, Gas and Electricity of New York City, to re-organize it along scientific lines, and to carry on its work in a business-like way. This has been done, it is said, and this Report shows what the conditions were before the re-organization and the results accomplished thereby. The Bureau, which was demoralized and inefficient, is to-day, it is stated, a model of its kind in its functional organization, its methods have been standardized, and its employes so instructed and trained as to make its present operation well-nigh automatic. An endeavor has been made to formulate clearly the fundamental principles involved in this work of re-organization and to present them so that this Report may serve as a textbook and guide to other officials, either in public service or in private business, who may be called on to undertake a similar task. The Chapter headings are: Plan and Scope; Organization and Administration; Purchasing; Inspection; Storage and Issue; Records and Accounts.

Gifts have also been received from the following:

Alabama-Geol. Survey. 1 pam.
Alabama-State Dept. of Archives and History. 1 bound vol.
Alabama-State Highway Dept. 1 pam.
Am. Soc. of Mech. Engrs. 1 bound vol.
Am. Telephone & Telegraph Co. 1 bound vol., 2 pam.
Am. Wood Preservers' Assoc. 1 bound vol., 1 pam.
Arizona-State Board of Equalization. 1 pam.
Attleborough, Mass.-Town Clerk. 1 vol.
Attleborough, Mass.-Water Dept. 1 pam.
Auburn, N. Y.-Water Board. 1 pam.

* Unless otherwise specified, books in this list have been donated by the publishers.

Baltimore County, Md.-Roads Engr. 1 pam.
Bengal, India-Irrig. Dept. 1 pam.
Binghamton, N. Y.-Water Commrs. 1 pam.
British Columbia-Dept. of Lands. 1 vol.
Brookline, Mass.-Water Dept. 1 pam.
Brown, Rome G. 1 pam.
Burlington, Vt.-Mayor. 1 bound vol.
Burlington, Vt.-Water Dept. 1 pam.
California-Board of State Harbor Commrs. 2 pam.
California-Secy. of State. 1 pam.
Cambrian Rys. Co. 1 pam.
Canada-Dept. of Mines. 13 vol., 2 pam.
Canada-Dept. of Railways and Canals. 1 vol., 3 pam.
Chosen-Director of the Railway Bureau. 1 pam.
Cleveland, Ohio-City Clerk. 1 bound vol.
Clinton, Mass.-Water Dept. 2 pam.
Colorado-Agri. Exper. Station. 3 pam.
Dartmouth Coll.-Thayer School of Eng. 2 pam.
Derby, England-Borough Development Comm. 2 pam.
Dover, N. H.-Board of Water Commrs. 1 pam.
Durham, H. W. 1 vol.
Electrician, The. 1 bound vol.
Essex County, Mass.-County Engr. 1 pam.
Florida-State Geol. Survey. 1 vol.
Great Central Ry. Co. 1 pam.
Great Western Ry. Co. 1 pam.
Green Bay & Western R. R. Co. 2 pam.
Grosvenor, A. W. 1 pam.
Hinds, Noble & Eldredge. 1 bound vol.
Humphreys, Benjamin G. 1 bound vol.
Illinois-Dept. of Factory Inspection. 2 pam.
Illinois-State Geol. Survey. 1 bound vol., 1 map.
Illinois Miners' and Mechanics' Institutes. 1 pam.
Illinois, Univ. of. 1 pam.
Indiana Univ. 1 vol.
Iowa-Geol. Survey. 1 bound vol., 1 pam.
Iowa-R. R. Comm. 1 bound vol.
Kansas-Insp. of Coal Mines. 1 pam.
Kansas-Public Utilities Comm. 1 vol., 1 pam.
Kansas, Univ. of. 2 pam.
Koehring Machine Co. 1 pam.
Laurgaard, Olaf. 1 pam.
Lehigh Coal & Navigation Co. 1 pam.
Lilly, W. E. 1 pam.
Liverpool, England-Town Clerk. 1 bound vol.
London & North Western Ry. Co. 1 pam.
Los Angeles, Cal.-City Auditor. 1 bound vol.
Louisiana-R. R. Comm. 1 map.
McLean, Joseph P. 4 bound vol., 7 pam.
Madras, India-Public Works Dept. 1 pam.
Maine-R. R. Comm. 1 bound vol.
Maine-State Treasurer. 1 bound vol.
Massachusetts-Harbor and Land Commrs. 1 vol.
Mellon Institute of Industrial Research and School of Specific Industries. 1 pam.
Michigan-Geol. and Biological Survey. 1 bound vol., 3 vol.
Midland Ry. Co. 1 pam.
Minneapolis, Minn.-Public Library. 1 bound vol.
Minnesota-State Board of Control. 1 pam.
Mississippi, Univ. of. 1 pam.
Missouri-Public Service Comm. 2 pam.
Missouri-Secy. of State. 2 pam.
Mobile, Ala.-Board of Commrs. 1 pam.
Montana-Board of R. R. Commrs. 1 bound vol.
Municipal Engrs. of the City of New York. 1 pam.
National Fire Protection Assoc. 4 pam.
Nebraska-State Ry. Comm. 1 vol.
Nevada-Insp. of Mines. 2 pam.
Nevada-Public Service Comm. 2 pam.
New Jersey-Public Library Comm. 1 pam.
New South Wales-Met. Board of Water Supply and Sewerage. 1 bound vol.
New York City-Dept. of Health. 4 pam.
New York City-Dept. of Highways. 3 pam.
New York State-Dept. of Health. 1 bound vol.
New York State-Metropolitan Sewerage Comm. 7 pam.
New York State-Public Service Comm., First Dist. 1 pam.
New York State-Supt. of Public Works. 3 bound vol., 1 vol.
New York City Record. 1 bound vol.
New York State Charities Aid Assoc. 1 pam.
New York Univ. 1 bound vol., 1 pam.
Newark, N. J.-City Plan Comm. 1 bound vol.
Newcastle & Gateshead Water Co. 2 pam.
Newcastle-upon-Tyne, England-City Engr. 1 pam.
Norske Ingeniorforening. 1 pam.
North British Ry. Co. 1 pam.
North Dakota-R. R. Commrs. 1 pam.
North-East Coast Institution of Engrs. and Shipbuilders. 1 bound vol.
Northampton, Mass.-Water Commrs. 1 pam.
Ohio-Public Utilities Comm. 1 pam.
Oklahoma-Corporation Comm. 1 bound vol., 1 map.
Oklahoma Soc. of Engrs. 3 pam.
Oregon-Bureau of Mines and Geol. 2 pam.
Oregon-R. R. Comm. 1 vol.
Oregon-State Board of Forestry. 1 pam.
Oregon-State Water Board. 1 pam.
Oregon Soc. of Engrs. 1 pam.
Pennsylvania R. R. Co. 1 pam.
Permanent International Assoc. of Navigation Congresses. 1 vol.
Philippine Islands-Bureau of Science. 1 pam.
Pittsburgh, Pa.-Carnegie Library. 1 pam.
Porto Rico-Direccion de Sanidad. 1 pam.
Providence, R. I.-City Engr. 1 pam.
Reading, Mass.-Water Commrs. 1 pam.
Referendex Co., Limited. 1 bound vol.
Rhode Island-State Board of Health. 1 bound vol.
Royal Philosophical Soc. of Glasgow. 1 vol.
R. Scuola d'Applicazione per gli Ingegneri in Roma. 1 pam.
Sacramento, Cal.-City Clerk. 1 pam.
Saint Pancras (London), England-Town Clerk. 1 bound vol.
San Francisco Chamber of Commerce. 1 pam.

Sandoval, Aurelio. 1 pam.
Schiffbautechnischen Gesellschaft. 1 bound vol.
Smithsonian Institution. 1 vol., 5 pam.
Soc. of Naval Archts. and Marine Engrs. 2 pam.
South Australia-Dept. of Public Works. 1 vol.
Southend Water-Works Co. 1 pam.
Stiles, Arthur A. 1 pam.
Sturtevant, B. F., Co. 1 bound vol.
Switzerland-Landeshydrographie. 3 pam.
Taunton, Mass.-Water Commrs. 1 pam.
Texas, Univ. of. 2 pam.
Titanium Alloy Mfg. Co. 3 pam.
Tyrrell, Henry Grattan. 1 bound vol.
U. S.-Bureau of Mines. 1 vol., 4 pam.
U. S.-Bureau of Standards. 6 pam.
U. S.-Bureau of the Census. 2 bound vol.
U. S.-Coast and Geodetic Survey. 2 bound vol.
U. S.-National Museum. 1 pam.
U. S.-Ordnance Dept. 1 bound vol.
U. S.-Reclamation Service. 2 pam.
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SUMMARY OF ACCESSIONS

(From March 3d to April 1st, 1914)

Donations (including 18 duplicates)............	249
By purchase..................................	11
Total..	260

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HALL, FREDERIC FRANCIS. 25 Hillside Court, Berkeley, Cal.		Dec. 31, 1913
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KERRY, JOHN GEORGE GALE. Pres., Kerry & Chace, Ltd., Confederation Life Bldg., Toronto, Ont., Canada		Mar. 4, 1914
KOON, RAY EMERSON. Hydr. and San. Engr., 875 East Yamhill St., Portland, Ore.	Assoc. M. M.	Oct. 7, 1908 Mar. 4, 1914
MILLIGAN, ROBERT EDGAR. Gen. Mgr., The New York Continental Jewell Filtration Co., 15 Broad St. (Res., 1 West 64th St.), New York City		Mar. 4, 1914
POPE, JAMES BOND. Asst. Engr., S. P. Co., 1043 Flood Bldg., San Francisco, Cal		Feb. 4, 1914
SMITH, CHESTER ALEXANDER. Chf. Engr., Burns & McDonnell, 823 Scarritt Bldg., Kansas City, Mo	Assoc. M. M.	April 5, 1910 Mar. 4, 1914
SPENCER, THEODORE NELSON. Asst. Engr., Dept. of City Transit, Room 744, City Hall (Res., 4903 Cedar Ave.), Philadelphia, Pa	Assoc. M. M.	Sept. 5, 1900 Mar. 4, 1914
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CHAPMAN, ASA B. 539 Pacific Elec. Bldg., Los Angeles, Cal.

DALY, ALBERT PETER VINCENT. 27 Manhattan Ave., New York City.

DAVENPORT, ROYAL WILLIAM. Asst. Engr., Water Resources Branch, U. S. Geological Survey, 421 Federal Bldg., Salt Lake City, Utah.

JUNIORS (*Continued*)

DIRATZOUYAN, JANIK KEVORK. Res. Sales Engr., American Levant Agency, Manoukian Han, Galata (*via* London), Constantinople, Turkey.

DONLE, EARL RAYMOND. Benjamin Rd., Belmont, Mass.

DORRANCE, FRANK YOUNG. Prin. Asst. Engr., Filtration Works, 805 Durocher St., Montreal, Que., Canada.

DRURY, WILLIAM FISHER. Care, Central Maine Power Co., Waterville, Me.

FEELEY, WILLIAM PATRICK. Care, Great Lakes Dredge & Dock Co., 1100 Morgan Bldg., Buffalo, N. Y.

HENNING, CHARLES SUMNER, JR. Idaho Falls, Idaho.

HITT, HENRY COLLINS. Asst. State Field Engr., Olympia, Wash.

KING, TAO. 11 Yen Chia Hong, Soochow, China.

LEARNED, ALBERT PREISACH. Res. Engr., with Worley & Black, Cons. Engrs., 301 Reliance Bldg., Kansas City, Mo.

LISMAN, OLIVER CROMWELL. Vice-Pres. and Gen. Mgr., Kansas City & Memphis Ry., Rogers, Ark.

MACKRELL, EDWIN ALLEN. 971 East 18th St., Brooklyn, N. Y.

MONROE, ROBERT ANSLEY. 1016 Eddy St., San Francisco, Cal.

MORGAN, JOSEPH HOLLOWAY. Junior Engr., U. S. Geological Survey, 1741 F St., N. W., Washington, D. C.

MOSER, ALBERT LEO BRECHT. 1125 Twenty-seventh St., Denver, Colo.

MUCHEMORE, HARRIE LANGDON. Engr., W. W. Anderson & Co., 107 Grant Ave., San Francisco (Res., 2843 Green St., Berkeley), Cal.

O'NEIL, HARRY BERNARD. Civ. Engr. and Surv., 61 Court St., Keene, N. H.

ROWE, WILFRED LINCOLN. Meadow Creek, Wash.

TIMBERLAKE, SETH MARTIN. Junior Engr., Public Service Comm., First Dist., 154 Nassau St., New York City (Res., 566 Park Ave., East Orange, N. J.).

WHITMORE, HAROLD CUSHING. P. O. Box 361, Shelburne Falls, Mass.

WILSON, CALVIN LOUGHRIDGE. 1222 South Jennings Ave., Fort Worth, Tex.

WILSON, WILLIAM WEST. 95 Montclair Ave., Newark, N. J.

WOLFF, ALFRED DANIEL, JR. Asst. Engr., N. Y. C. & H. R. R. R., 22 Summit Ave., White Plains, N. Y.

YEREANCE, ALEXANDER WOODWARD. 418 Centre St., South Orange, N. J.

FELLOW

HILL, JAMES JEROME. 291 East 3d St., St. Paul, Minn.

RESIGNATIONS

ASSOCIATE MEMBERS	Date of Resignation.
HOWE, CLARENCE DURAND	Mar. 4, 1914
KOENIG, ARNOLD CHARLES VON WASMER	April 1, 1914

ASSOCIATE

AMBLER, DANIEL GRIFFITH	Mar. 4, 1914

DEATHS

HOLLOWAY, ROGER TIFFT. Elected Junior, May 31st, 1910; Associate Member, May 7th, 1913; died March 13th, 1914.

HUNICKE, WILLIAM AUGUST. Elected Associate Member, March 2d, 1904; Member, January 7th, 1913; died March 9th, 1914.

JENKINS, WILLIAM DUNBAR. Elected Member, September 7th, 1887; died March 12th, 1914.

MORSE, BENJAMIN FRANKLIN. Elected Member, July 12th, 1877; died February 24th, 1914.

WESTINGHOUSE, GEORGE. Elected Member, January 7th, 1891; died March 12th, 1914.

Total Membership of the Society, April 2d, 1914,
7 377.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(March 2d to April 1st, 1914)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., St. Louis, Mo., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.

(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England, 6d.
(69) *Der Eisenbau*, Leipzig, Germany.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Mining and Engineering World*, Chicago, Ill., 10c.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga., 10c.
(109) *Journal*, Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal*, Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas*, San Francisco, Cal., 25c.

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Bridges.

The Erection of the Boucanne River Viaduct, Canada.* Philip Louis Pratley. (63) Vol. 193.
The Strengthening of Wrought-Iron Bridges on the Cape Government Railways.* James Mackenzie. (63) Vol. 193.
Bridges on the Northern Section of the Nagdamuttra State Railway.* John Kerr Robertson. (63) Vol. 193.
The Design and Construction of the Chicago Great Western Ry. Bridge Over the Mississippi River at St. Paul, Minn.* (86) Dec. 10.
The Demolition of the Old Southwark Bridge.* (11) Feb. 27.
Rolling Loads on Railway Girder Bridges.* F. C. Lea. (11) Serial beginning Feb. 27.

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Bridges—(Continued).

Protection of Girders from Locomotive Smoke.* H. K. McCay. (60) Mar.
Construction of the Fallsway Viaduct, Baltimore, Md. Louis R. Gons. (36) Mar.
Novel Type of Reinforced Concrete Abutment.* Alfred W. Hoffman. (87) Mar.
Through Arch Bridges of Reinforced Concrete.* A. M. Wolf. (87) Mar.
Progress on the Haights Run Arch, Pittsburgh.* (13) Mar. 5.
Some Features of the Design and Construction of the City Waterway Bridge at Tacoma, Wash.* (86) Mar. 11.
Repairing a Teredo-Eaten Bridge Pier Foundation. (14) Mar. 14.
Fabrication of Quebec Bridge Members, the Largest and Heaviest Ever Built.* (14) Mar. 14.
The Langwies Arch.* H. Schuerch. (13) Mar. 19.
Ultimate Strength of Carbon and Nickel-Steel Models of Quebec Bridge Members.* (14) Mar. 21.
Design of the 531-Ft. Truss Spans of the North Side Point Bridge, Pittsburgh, Pa.* (86) Mar. 25.
The Field Work of the Lethbridge Viaduct.* B. Ripley. (96) Mar. 26.
C. N. O. R. Bridge Over East Sturgeon River.* (96) Mar. 26.
Scherzer Rolling Lift Bridge for the Indo-Ceylon Connection.* (18) Mar. 28.
Progress at the Site of Quebec Bridge.* (14) Mar. 28.
Note sur le Calcul de Quelques Cas Particuliers de Ponts à Béquilles.* R. Despret. (30) Feb.
Equipment Electrique du Pont à Bascule d'Engelburg à Dordrecht (Hollande).* O. H. Wildt. (33) Feb. 21.
Passerelle Suspendue de 135 Mètres de Portées Reliant les Usines des Etablissements Arbel, à Douai.* G. Leinekugel Le Cocq. (33) Feb. 28.
Die neue Kaiser-Wilhelm-Brücke über die Spree in Fürstenwalde.* Mensch. (40) Feb. 11.
Ueber die Anwendung der Ritzschen Methode zur Berechnung eines Kuppelgewölbes.* A. Leon und P. Fillunger. (53) Serial beginning Feb. 27.
Neubau der Streekbrücke in Hamburg.* Schwoon. (78) Mar. 17.

Electrical.

Apparatus for Duplex Telegraphy.* Ricardo Lopez. (Abstract translation from *Revista Telegrafica.*) (73) Jan. 9.
Quenched Spark Wireless Telegraphy.* W. T. Ditcham. (73) Jan. 9.
The Loading of Aerial Lines and Their Electrical Constants. J. G. Hill. (Abstract of paper read before the Institution of Post Office Elec. Engrs.) (73) Jan. 16.
The Use of Telegraph Lines for Long-Distance Telephony.* Bela Gati. (73) Serial beginning Jan. 16.
The Tone-Wheel Detector for Wireless Telegraphy.* Rudolf Goldschmidt. (73) Jan. 16.
Overhead Wire Construction for Medium and Low Pressures.* A. P. Trotter. (Abstract of paper read before the Institution of Post Office Elec. Engrs.) (73) Jan. 23.
Telephone Transmission. Bancroft Gherardi. (Abstract of paper read before the New York Telephone Soc.) (73) Jan. 30.
Investigation on the Temperature of Field Coils.* E. H. Rayner. (Paper read before the Inter. Electrochemical Comm.) (73) Jan. 30.
Laying Telephone Cables Across the Golden Horn, Constantinople.* A. Podmore. (73) Jan. 30.
Modern Street Illumination. Charles L. Kinsloe. (98) Feb.
Moderate Capacity Outdoor High Tension Substations.* H. W. Young. (4) Feb.
Tariffs Dependent on Power Factor. R. Arno. (From papers read before the Inter. Congress at Turin.) (73) Feb. 6.
The Use of High-Frequency Alternating Currents in Telegraphy, Telephony and for Power Transmission.* A. Maior. (73) Feb. 6.
Design of the Radio-Telegraph Transmitter. A. S. Blatterman. (73) Serial beginning Feb. 13.
The Betulander Automatic Telephone System.* (12) Feb. 20.
Some Proposed Improvements in the Wheatstone Automatic System.* Ricardo Lopez. (Translation from the *Journal Telegraphique.*) (73) Feb. 20.
Siemens' Automatic Telephone System. (11) Feb. 27.
Measurement of the Average Value of the Answering Time at Odense Exchange, Denmark.* J. von Linstow. (73) Feb. 27.
Automatic Telephones at King's College Hospital, London.* (73) Feb. 27.
The Interpretation of Electric Current Flow in Terms of the Electron Theory. W. F. Durand. (111) Feb. 28.
The Economical Capacity of a Combined Hydro-Electric and Steam Power Plant.* Cary T. Hutchinson. (42) Mar.
The Cost of Electricity at the Source. H. M. Hobart. (42) Mar.
Recording Devices. Charles P. Steinmetz. (42) Mar.
Comparison of the Telegraph with the Telephone as a Means of Communication in Steam Railroad Operation. M. H. Clapp. (42) Mar.

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Traffic Studies in Automatic-Switchboard Telephone Systems.* W. Lee Campbell. (42) Mar.
Voltage Regulation in Electric Distribution System.* G. G. Post. (60) Mar.
Sixty-Cycle Synchronous Converters. L. P. Crecelius. (42) Mar.
The Present Physical Knowledge of X Rays. Wheeler P. Davey. (3) Mar.
Electric Gantry Cranes for Terminals.* F. W. Buse. (95) Mar.
The Electrification of the Atmosphere, Natural and Artificial.* Oliver Lodge. (77) Mar. 2.
Electrical Pyrometry.* (47) Serial beginning Mar. 6.
The Electrical Equipment of the R. M. S. *Britannic*. (73) Mar. 6.
Electrical Distribution Engineering in Chicago.* (27) Serial beginning Mar. 7.
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Electric Power Supply at Sheffield.* (73) Mar. 13.
The Wet Filtration of Cooling Air for Electrical Machinery. D. A. Hackett. (26) Mar. 13.
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Transmission Plant Without a Switchboard.* Louis Bell. (27) Mar. 14.
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The Gott System of Telegraphing Through Long Submarine Cables.* (26) Mar. 20.
Reinforced-Concrete Poles withstand Blizzard, Heavy Snow and Sleet Storm in Vicinity of New York Breaks Wooden Poles and Interrupts Telephone and Telegraph Communication.* (14) Mar. 21.
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Electricity and Matter. Henry S. Carhart. (111) Mar. 21.
Steam and Hydroelectric Generating System.* (27) Mar. 28.
Donaldsonville, La., Electric-Light and Water-Works Plant.* E. M. Ivens. (64) Mar. 31.
Discrimination in Rates for Electricity.* Henry D. Jackson. (64) Mar. 31.
Turbo-Alternateur Parsons de 25 000 Kilowatts, de la Commonwealth Edison Co., de Chicago.* (33) Mar. 7.
Die elektrische Küche im Grossbetrieb.* W. Schulz. (7) Feb. 14.
Elektrisch betriebener, in Güterzüge einstellbarer Drehkran für Greiferbetrieb.* E. Borghaus. (102) Feb. 15.
Das Marmorlicht.* W. Voege. (41) Feb. 19.
Die neue Fernleitungsstelle in München.* Wilhelm Schreiber. (41) Serial beginning Feb. 26.
Versuche über die Beeinflussung des Wachstums der Pflanzen durch Elektrizität.* Herbert G. Dorsey. (41) Feb. 26.
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Ueber Stosssender der drahtlosen Telegraphie.* Eugen Nesper. (41) Serial beginning Mar. 19.
Die Elektrizitätsversorgung in Bayern.* F. X. Gebele. (41) Mar. 19.
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The Navy and the Panama Canal. Harry S. Knapp. (From *Proceedings*, United States Naval Inst.) (44) Mar.
The Strategic Aspect of the Panama Canal.* Vaughan Cornish. (44) Mar.
Vulcan Marine Steam Turbines.* Alfred Gradenwitz. (64) Mar. 3.
Life Saving Appliances. John Harvard Biles. (Paper read before the North-East Coast Institution of Engrs. and Shipbuilders.) (11) Serial beginning Mar. 6.
The Motor Ship *Sebastian*.* (12) Mar. 6.
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The Strength of Stayed Flat Plates. C. E. Stromeyer. (Paper read before the Manchester Steam Users' Assoc.) (12) Mar. 13.
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A New Method of Coaling Gas-Engines. Bertram Hopkinson. (75) July.
Strength of Equipment for Handling Loads.* Edward Godfrey. (58) Jan.
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Carbonization in Bulk for Gas Production.* G. Stanley Cooper. (Paper read before the London and Southern Junior Gas Assoc.) (66) Feb. 24.
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Handling Steamship Package Freight.* Williard C. Brinton. (95) Mar.
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The Economical Handling of Package Freight and General Cargo on Steamship Docks.* C. A. Hardy. (95) Mar.
The Manufacture and Uses of Blaugas. Hugo Lieber. (105) Mar.
Economical Lubrication. W. W. Davis. (Paper read before the Lake Superior Min. Inst.) (45) Mar.
On the Theoretical Efficiency of the Linde Process of Liquefying Air. M. M. Garver. (3) Mar.
High Pressures and Five Kinds of Ice. P. W. Bridgman. (3) Mar.
Standardizing Machinery. Fred H. Colvin. (55) Mar.
Standardization of Pipe Thread Gages. (Report of Committee, Am. Soc. of Mech. Engrs.) (55) Mar.
Standard Threads for Hose Couplings.* (Report of Sub-committee on Fire Protection, Am. Soc. of Mech. Engrs.) (55) Mar.
Power from Mercury Vapor. W. L. R. Emmet. (42) Mar.
Comparative Tests of Three Types of Bearings.* Carl C. Thomas, E. R. Mauer and L. E. A. Kelso. (55) Mar.
Modernizing a Southern Lime Mill.* Ellis Soper. (Paper read before the Am. Assoc. for the Advancement of Science.) (67) Mar.
High-Pressure Gas for Street Lighting. T. Sington. (60) Mar.
Pulverized Coal in Malleable Plant.* (62) Mar. 2.
Experiences in Automobile Operation.* Geo. P. Smith, Jr. (Paper read before the New England Assoc. of Gas Engrs.) (83) Mar. 2.
Comparison of Joints for Cast-Iron Mains.* H. B. Coho. (Paper read before the Am. Gas Inst.) (83) Mar. 2.
Vertical Retort Settings in English Works. C. A. Learned. (Paper read before the New England Assoc. of Gas Engrs.) (83) Mar. 2; (24) Mar. 2.
Gas-Producer Operations.* E. G. Moon. (64) Mar. 3.
Chimney Capacity for Burning Waste Fuels. David Moffat Myers. (64) Mar. 3.
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Steam-Raising by Gas-Coke. E. W. L. Nicol. (66) Mar. 3.
The Running of Coal and Coke-Handling Machinery as Applied to Horizontal Retorts. W. H. Adams. (Paper read before the Midland Assoc. of Gas Mgrs.) (66) Mar. 3.
Building Large Gas Engine Flywheels.* J. H. B. Bryan. (20) Mar. 5.
Electrical Pyrometry.* (47) Serial beginning Mar. 6.
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Elements of Fuel Oil Practice and Boiler Testing.* Robert Sibley. (111) Serial beginning Mar. 7.
Recommendations for Standard Specifications, Horizontal Water Tube Boilers. Thomas E. Durban. (Paper read before the National Assoc. of Tubular Boiler Mfrs.) (62) Mar. 9.
Remodeled Railway, Electric and Ice-Making Plant at Hampton, Va.* Warren O. Rogers. (64) Mar. 10.
Some Results of Twenty-Four Hour Plant Test.* B. R. T. Collins. (64) Mar. 10.
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A Diesel Engine-Driven Air-Lift Pumping Plant.* (12) Mar. 13.
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The Biggest of French Dirigibles.* Carl Dienstbach. (46) Mar. 14.
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Standard Street Main Practice. G. I. Vincent. (Paper read before the Am. Gas Inst.) (83) Mar. 16.
Improvements in English Gas Producers.* Arthur H. Allen. (64) Mar. 17.

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Working Up a Boiler Test. F. R. Low. (64) Mar. 17.
Applications of Combined Swing Gate and Check Valves.* Oscar C. Schmidt. (64) Mar. 17.
A 10 000-Ton Hydraulic Forging Press.* C. von Philip. (20) Mar. 19.
Small Steel Castings as a Specialty. C. S. Koch. (Paper read before the Pittsburgh Foundrymen's Assoc.) (20) Mar. 19; (62) Mar. 16.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XL. APRIL, 1914. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE

By Charles H. Lee, Assoc. M. Am. Soc. C. E.

To be Presented May 6th, 1914.

Summary and Conclusions.

The objects of this paper are to show the possibility and practicability of measuring the annual rate of recharge of underground reservoirs of the closed-basin type, and to indicate broadly the factors which determine safe yield from a basin by artificial development, such as Artesian flow or pumping.

The paper opens by pointing out the importance of the problem in California and the Southwest. Following this there is a description of the physical features of underground reservoirs and the general principles of inflow, outflow, and storage. The body of the paper presents detailed methods and results of extended measurements, by the Los Angeles Aqueduct Bureau and the United States Geological Survey, for the determination of the rate of annual recharge of the Independence Basin, in Owens Valley, California. The subjects of percolation from stream channels, relation of precipitation and altitude, soil evaporation combined with transpiration from grass, and ground-water fluctu-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

ations, were carefully studied in the field, and original data are presented. The paper closes with a discussion of the relation which the net safe yield from a basin bears to the rate of annual recharge.

The conclusions are as follows:

1.—The "underground reservoirs" of California and the Southwest are water-tight rock basins, represented by the topographic valleys, which are filled with porous alluvial material in which the voids are saturated with water.

2.—Inflow into these basins is by percolation from water on the surface of the alluvial filling, which source may occur as direct precipitation, stream flow, irrigation, or flooding. Natural ground-water loss occurs in the region of lowest depression of a basin, and consists of the breaking out of water at the surface in springs or seepages, evaporation from soil, transpiration, and underflow. Artificial development, by wells or other methods, reduces the natural ground-water loss. Considered as averages, the rates of recharge and ground-water loss are equal, unless the artificial draft is excessive.

3.—The rate of recharge in a region of small precipitation and high evaporation rate can be determined most accurately, and with least expenditure of time and money, by measuring the elements which make up the ground-water loss. Of the natural elements, the most important are soil evaporation and transpiration. The underflow is relatively small and often negligible.

4.—The safe yield of artificially developed ground-water obtainable from an underground reservoir is less than indicated by the rate of recharge, the quantity depending on the extent to which soil evaporation and transpiration can be eliminated from the region of ground-water outlet.

Introduction.

There are in California and the arid States of the Southwest many valleys underlaid by porous alluvial material in which the voids are filled with water. The ease with which water can be developed from wells in these valleys and the definite bounds of the water-bearing formation have led to the use of such terms as "underground lake," "underground basin", or "underground reservoir". These terms are in general use among local hydraulic engineers, and have been adopted by the California Courts in numerous recent decisions pertaining to the use of diffused percolating water occurring in closed basins.

A problem which is being presented to the Engineering Profession for solution is the determination of the safe yield of "underground reservoirs", or the net annual supply which may be developed by pumping and Artesian flow without persistent lowering of the ground-water plane. The answer to this problem must soon be had throughout the Southwest, and particularly in Southern California, where the use of underground water has advanced most rapidly. The available surface supplies of the region are now used so extensively that future extension of irrigation must depend on the underground supply. Already, however, the growing popularity of ground-water supply for irrigation and the heavy drafts made possible by improved pumping machinery and cheap power are giving rise to conditions of dangerous overdraft on many of the so-called inexhaustible underground water supplies. Furthermore, in many of the sparsely settled valleys of the Southwest, where very limited ground-water supplies are available, preparations are being made to develop pumped water for irrigation far in excess of the safe yield. The writer has in mind such a valley, where, out of 90 000 acres of agricultural land, filed on in good faith under the provisions of the Desert Land and Homestead Acts, it can be said with reasonable certainty that not more than 2% can ever be put under cultivation. In addition to the use of underground water for irrigation, it is being developed extensively for municipal purposes. The City of Los Angeles derives its present supply entirely from an underground reservoir, the San Fernando Valley, and is preparing to develop a similar supply in Owens Valley to be held as a reserve in connection with the Los Angeles Aqueduct. A portion of the supply of both Oakland and San Francisco is developed from underground sources, and the possibility of increasing largely the ground-water supply derived from Livermore Valley for the latter city has been the subject of considerable debate among prominent members of the Society. The problem, therefore, is an important one, and on it depends, not only the safe investment of capital, but also the very life of large industries and communities.

The sources of underground water are so difficult of measurement and its movements are so hidden from view, that the solution of the problem, until very recently, has been merely a subject for speculation and theory. Within the past few years, however, the study of underground water supply has been given considerable attention by the

United States Geological Survey as well as by engineers who have had these problems to meet. Although the fact of the existence of these "underground reservoirs" has been established, their sources of supply and outlets recognized, and many data regarding well fluctuations have been accumulated, yet very little has been done toward developing methods of measuring the rate of recharge or studying the factors which limit the quantity of water which can be safely developed from underground reservoirs.

The writer has had opportunity to investigate a number of the important underground reservoirs of California, and in this paper he presents certain general principles which seem to him to be justified by the existing data. Although these principles may seem to be self-evident, yet the writer has no knowledge that they have ever been applied to the practical solution of the problem in hand. To show the possibilities of their application, therefore, he presents data and studies for an underground reservoir in Owens Valley, California, where it was desired to ascertain the quantity of ground-water that could be developed safely without overdraft. Much of this information has already appeared in print* in greater detail, but the writer believes that the subject is of sufficient importance, and the component studies are of wide enough technical interest to be presented to the members of the Society for discussion and expression of opinion.

General Principles.

The typical underground reservoir is, geologically, a structural basin filled with alluvial débris from the adjoining mountain ranges. These basins are the product of faulting accompanied by the uptilting of a crustal block from one side of the line of fracture. The formation is very common throughout the Southwest, reaching its most perfect development in the Great Basin region of Utah and Nevada, where the name "Basin-Range" has been applied to it. In California the basins are found in the valleys of the Coast Range and along the base of the Sierra Nevada, Sierra Madre, San Bernardino, and San Jacinto Ranges. The rock enclosing these basins is in most cases impervious to water and practically insoluble. Along the coast of California, shales and cemented gravels predominate, and are practically non-water-bearing in comparison with the porous gravels

* Water Supply Paper No. 294, U. S. Geological Survey, 1912.

filling the basins; and, in the interior of the State, the enclosing rock formation is largely granite. Most of the basins can be considered as closed except for a subterranean outlet usually known as the "Narrows". This occurs at the lowest point in the rock rim, where the gravels contract into a neck filling a narrow depression or canyon cut into the confining rock. The quantity of underground water escaping through such an outlet is usually very small, however, as has been shown by a number of well-known underflow observations. Hence, the underground reservoirs can generally be considered as closed rock basins, the effective storage capacity of which is the void spaces between the particles of sand and gravel with which they are filled.

The usual sources of supply for underground reservoirs are percolation from flowing surface streams, from precipitation, or, where the supply is not ground-water derived from the basin, from irrigation on the surface of the porous gravels. The water thus absorbed sinks downward to the general ground-water plane and then moves laterally toward the region of lowest depression. This region, in contrast to the surrounding dry soil or desert, is usually characterized by springy, swampy conditions, and is commonly known in Southern California as a cienaga. The natural outlets for underground water are by springs or seepages discharging into the surface channels which drain the cienaga, by evaporation from damp soils and vegetation within the cienaga, and, to a limited extent, by underflow from the basin. The surface streams formed by the oozing out of underground water join to form a larger stream, which in all respects corresponds to the outlet of a lake or reservoir, and, passing from the basin, pursues its course just as any other surface stream. Its flow is characterized by permanence and regularity, except as it is augmented by surplus flood water which, during a limited period following winter storms, passes from the basin without being absorbed by the gravels.

The general principles of inflow into and outflow from an underground reservoir of the type described correspond with those of surface reservoirs. The difference lies in the relative speeds with which the general water surface assumes a horizontal position following increase or decrease of volume stored. In the case of a surface reservoir or lake, the effect of inflow or outflow is an immediate complete readjustment of surface level. The frictional resistance offered by the

particles filling an underground reservoir is so great, however, that the movement of water from an area of high level is very slow, varying from a few hundred feet to a few feet per day, depending on local conditions. As a result, the water surface in an underground reservoir is never horizontal, being steepest near the mouths of the mountain canyons, the run-off from which is the most important source of supply; it is most nearly horizontal at the region of outlet; and varies in slope and elevation from time to time, depending on the rate of recharge.

The average rates of inflow and outflow of an underground reservoir must be equal, otherwise there would be persistent rise or fall of ground-water levels until such a balance is reached. There are, therefore, two possible methods of measuring the rate of recharge, either by determining the total percolation from various sources into the porous material of the basin, or by determining the ground-water losses. The first method is to be preferred where the source of percolation is almost entirely stream flow from which channel losses can be accurately measured; or where the precipitation is large, well distributed through the year, and forms the principal source of supply. The first of these conditions could occur only in an arid region, and the second is typical of humid regions.

The method by determination of ground-water losses is one peculiarly adapted to arid or semi-arid conditions with high evaporation rate, such as exist throughout the Southwest. It has been the writer's observation in this region that soil evaporation and transpiration constitute from 50 to 100% of the ground-water losses from underground reservoirs, the average exceeding 75 per cent. Other losses are largely the flow from springs and seepages, which can be measured with precision. Rates of soil evaporation and transpiration from grasses do not present insurmountable difficulties of measurement under arid conditions. In fact, it has been the writer's experience that satisfactory results with specially designed equipment could be obtained from observations extending over 2 years, although a period of 3 years is preferable. Furthermore, the area from which evaporation occurs and the depth to ground-water at various points within it are not subject to wide fluctuations, and are easily measured. The determination of the rate of recharge of underground reservoirs of the basin type, therefore, is a problem of soil evaporation, transpiration, and stream

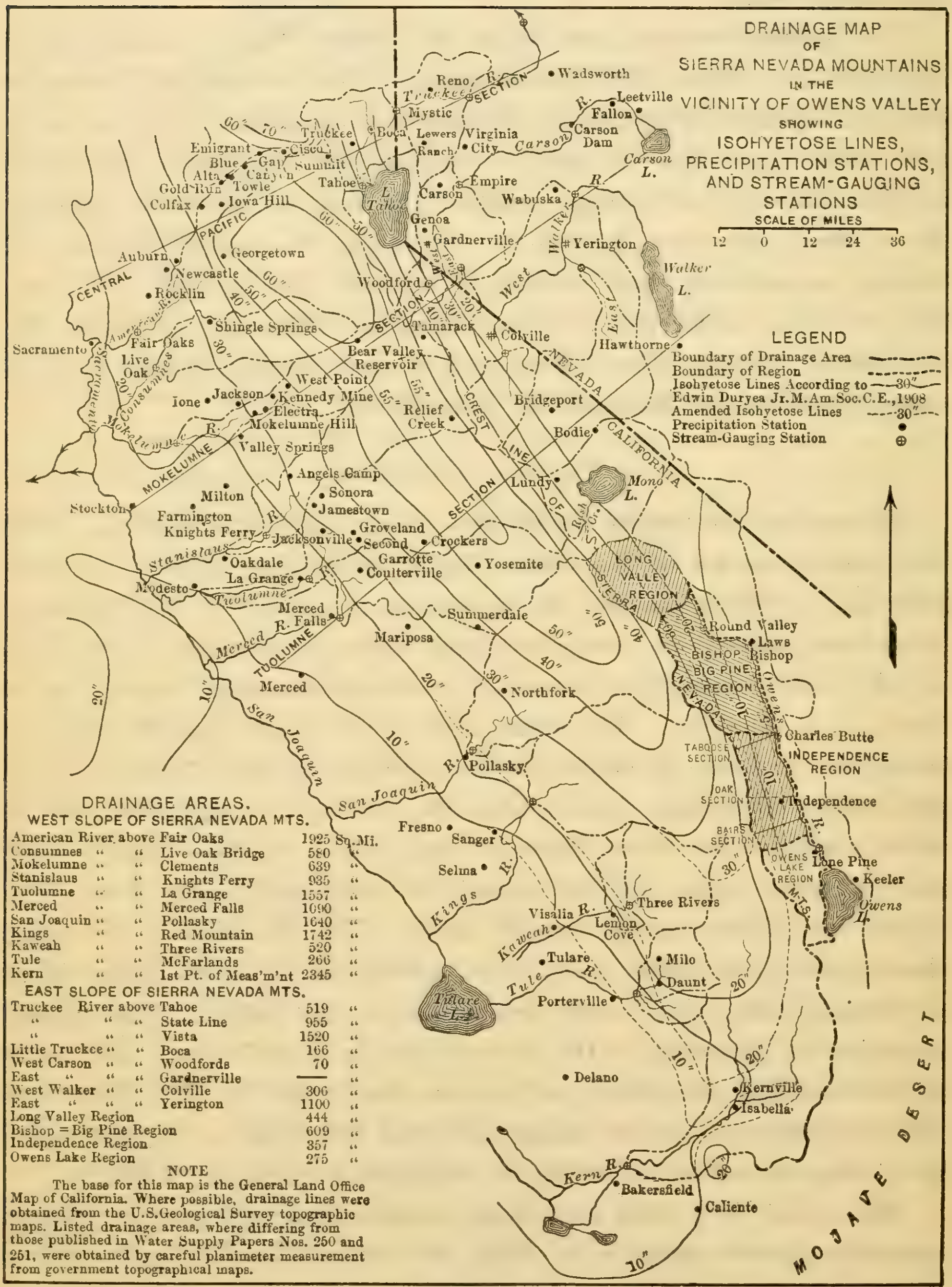
flow, all of which processes, with the exception of transpiration from trees, are now capable of measurement with relative accuracy at reasonable cost.

The general method pursued in the Owens Valley studies was to ascertain, by extended field measurements of soil evaporation, transpiration, and spring discharge, the average rate of outflow from the basin. All available evidence seemed to indicate that the basin was closed, so that the rate of outflow equalled the rate of inflow or recharge. As a check, therefore, the rate of inflow into this basin from precipitation, stream flow, and irrigation was also determined. The data are presented under the following headings: Physical Features, Precipitation, Stream Flow, Evaporation and Transpiration, Groundwater, and Rate of Recharge by Percolation.

Physical Features.

General.—The Owens Valley lies in east-central California, along the western border of the Great Basin, and at the base of the steep slope of the Sierra Nevada Mountains, as shown by Fig. 1. Including a northern extension, known as Long Valley, its length is 120 miles, and its width, from crest to crest of confining mountain ranges, varies from 15 to 40 miles. The total area of the valley and its tributary mountain drainage is about 3 300 sq. miles, of which 1 200 sq. miles are desert mountains from which the run-off is negligible, 536 sq. miles comprise the Sierra Nevada slope, which yields a large run-off, and 1 580 sq. miles are the transition slope and valley floor, from which very slight surface run-off occurs. The elevation of the valley floor varies from 8 000 to 3 570 ft. above sea level, the latter being at Owens Lake, the lowest depression of the valley. The average elevation of the crest of the Sierra Nevada is 12 500 ft., with many peaks exceeding this elevation by more than 1 500 ft. The White and Inyo Mountains, a desert range bordering the valley on the east, have an average elevation of 10 000 ft., with peaks reaching 13 000 ft.

The valley is a deep structural trough filled with porous alluvial material derived principally from the Sierra Nevada, and inclosed by impervious rock formations. The steep east face of the Sierra Nevada is the result of faulting accompanied by elevation and westward tilting of a great crusted block. The drainage system of the valley consists of a trunk stream, Owens River, fed by approximately



DRAINAGE AREAS.

WEST SLOPE OF SIERRA NEVADA MTS.

American River	above	Fair Oaks	1925 Sq. Mi.
Consumnes	" "	Live Oak Bridge	580 "
Mokelumne	" "	Clements	639 "
Stanislaus	" "	Knights Ferry	935 "
Tuolumne	" "	La Grange	1557 "
Merced	" "	Merced Falls	1090 "
San Joaquin	" "	Pollasky	1640 "
Kings	" "	Red Mountain	1742 "
Kaweah	" "	Three Rivers	520 "
Tule	" "	McFarlands	266 "
Kern	" "	1st Pt. of Meas'm'nt	2345 "

EAST SLOPE OF SIERRA NEVADA MTS.

Truckee	River above	Tahoe	519 "
"	" "	State Line	955 "
"	" "	Vista	1520 "
Little Truckee	" "	Boca	166 "
West Carson	" "	Woodfords	70 "
East "	" "	Gardnerville	— "
West Walker	" "	Colville	306 "
East "	" "	Yerington	1100 "
Long Valley Region			444 "
Bishop = Big Pine Region			609 "
Independence Region			357 "
Owens Lake Region			275 "

NOTE

The base for this map is the General Land Office Map of California. Where possible, drainage lines were obtained from the U.S. Geological Survey topographic maps. Listed drainage areas, where differing from those published in Water Supply Papers Nos. 250 and 251, were obtained by careful planimeter measurement from government topographical maps.

FIG. 1.

forty tributaries entering at fairly regular intervals from the west. The river terminates in Owens Lake, an alkaline body of water without outlet which disposes of all surplus surface water from the valley by evaporation. Irrigation is necessary for the production of crops, and water is diverted in canal systems from Owens River and tributary streams.

Independence Basin.—The portion of Owens Valley which is the subject of this study will be spoken of as the Independence Basin (Plate II). It lies in the south-central part of the valley, embracing with its tributary drainage area the region between the Sierra Nevada crest and Owens River, lying between Poverty Hills on the north and Alabama Hills on the south. These latter are secondary ranges extending into the valley from the Sierra Nevada and isolating a portion of the valley fill about 25 miles long and from 6 to 8 miles wide.

For the purpose of this study, the surface of the region has been classified as high mountain drainage, intermediate mountain slopes, outwash slope, and valley floor. (Table 1 and Plate II.)

TABLE 1.—SUBDIVISIONS OF INDEPENDENCE REGION.

Subdivision.	Area.		Boundary.		Slope.	Vegetation.	Character of surface.
	Square miles.	Percentage of total.	Upper.	Lower.			
High mountain drainage.	96.0	27	Sierra crest.	Mouth of canyon.	Precipitous to gentle.	Isolated forest trees.	Bare granite and fragmental rock accumulations.
Intermediate mountain slope.	29.4	8	Canyon drainage.	6 500-ft. contour.	2 000 to 3 000 ft. to the mile.	Desert bushes and nut pine.	Fragmental and finely disintegrated rock accumulations.
Outwash slope.	165.3	46	6 500-ft. contour.	Grass land.	300 to 600 ft. to the mile.	Desert bushes.	Boulders, sand and gravel.
Valley floor:							
Cultivated....	4.7	1	....	....	Gentle.....	Alfalfa, etc....	Soil.
Meadow.......	45.1	13	....	....	Gentle to level.	Salt grass, etc.	Soil.
Alkali.........	2.7	1	....	....	Level......	None...........	Soil.
Desert..	13.9	4	....	....	Level......	Desert bushes.	Fine sand.
	357.1	100					

The high mountain drainage embraces the eastern slope of the Sierra Nevada and consists of a series of seventeen small canyons which are the drainage basins of streams tributary to Owens River (Table 2). These canyons are all narrow at the mouth (the 6 500-ft. level) and broaden out more or less toward the summit, presenting a roughly triangular shape. They are separated by high knife-edge ridges, which terminate in triangular slopes facing the valley. They have been cut by water erosion and sculptured by active glaciation above the 7 500-ft. level, their upper portions being well-developed glacial cirques. In many places below the cirques are series of benches occupied by glacial lakes or meadows. Most of the cirque floors are buried beneath morainal accumulations; some of the polished canyon bottoms between the 11 500 and 8 000-ft. levels are swept clean of débris, and others are completely buried by morainal material. Terminal and lateral moraines of considerable size occupy the canyon floors between the 8 000 and 7 000-ft. levels.

TABLE 2.—HIGH MOUNTAIN DRAINAGE AREAS OF INDEPENDENCE REGION.

Creek.	Area.		Elevation.		Shape.	Length of Sierra crest drained, in miles.	Remarks.
	Total, in square miles.	Percentage above 10 000 ft.	Head of canyon, in feet.	Mouth of canyon, in feet.			
Taboose..........	7.16	60	12 000	6 500	Triangular...	3.34	Morainal deposits; regulated run-off.
Goodale..........	4.97	69	12 500	6 500	Triangular...	2.67	Morainal deposits; regulated run-off.
Dry Canyon......	2.48	65	12 000	6 500	Triangular...	1.21	Morainal deposits; no surface run-off.
Division..........	3.88	51	12 000	6 000	Rectangular.	1.88	Morainal deposits.
Sawmill...........	7.64	44	12 000	5 000	Rectangular.	2.75	
Thibaut (N. Fk.).	2.25	20	11 500	6 000	Irregular....	0.0	
Thibaut (S. Fk.)..	2.62	55	12 000	6 000	Irregular....	0.0	
Oak (N. Fk.).....	8.08	65	12 500	6 000	Irregular.....	5.17	Morainal deposits; regulated run-off.
Oak (S. Fk).......	7.28	57	12 500	6 000	Irregular....	1.06	
Little Pine........	8.42	74	13 000	6 500	Triangular...	4.60	
Pinyon............	4.29	47	13 000	6 500	Irregular....	1.09	
Symmes..........	4.22	43	13 000	6 300	Triangular...	1.59	
Shepard..........	12.29	66	13 500	6 500	Triangular...	7.95	5.68 miles of crest south of Kings-Kern divide.
Bairs (N. Fk.)....	4.01	43	13 500	6 300	Irregular....	0.0	Lies on east face of Mount Williamson.
Bairs (S. Fk.)....	2.90	41	13 000	6 300	Irregular....	0.0	Lies on east face of Mount Williamson.
George...........	9.10	74	13 500	6 500	Triangular...	3.89	
Hogback..........	4.38	58	13 000	7 000	Irregular....	0.67	
	95.97	55	......	......	....	37.87	


MAP OF INDEPENDENCE REGION
SHOWING
DRAINAGE AREAS, STREAM CHANNELS
AND PRECIPITATION STATIONS.
SCALE OF MILES
1.0 0.5 0 1 2 3 4 5
LEGEND.
Boundary of Mountain Drainage Areas
Boundary of Region
Isohyetose Lines
Stream Gauging Station Perennial Flow
Stream Gauging Station Intermittent Flow
Precipitation Station
West Boundary Valley Floor
BASE OF INYO MTS.
Owens River
VALLEY FLOOR
Los Angeles Aqueduct
Stevens Ditch
Southern Pacific System (Nevada and California R.R.) (Narrow Gauge)
Independence
Alabama Hills
Poverty Hills
Red Mountain
Charlies Butte
Goose Lake
LINE OF SIERRA NEVADA MTS.
Kings River Drainage
Kings-Kern Divide
Kern River Drainage

The intermediate mountain slopes (Fig. 2) are the triangular areas terminating the ridges between the canyons, and probably represent the original face of the range before it had been actively eroded (Table 3). Their lower boundary has been arbitrarily placed at the 6 500-ft. contour, and their apexes reach a maximum elevation of about 12 000 ft. They have a steep uniform slope of from 2 000 to 3 000 ft. to the mile, and in general are covered with a mantle of disintegrated rock and slide material which merges into the valley fill.

TABLE 3.—INTERMEDIATE MOUNTAIN SLOPES OF INDEPENDENCE REGION.

Adjoining high mountain drainage areas.	Area, in square miles.	ELEVATION.			Distance from Sierra crest to center of area, in miles.	Remarks.
		Apex, in feet.	Center of area, in feet.	Lower border, in feet.		
Tinemaha	2.17	(11 000)	8 000	6 500	3.0	
Red Mountain	2.37	(12 000)	8 300	6 500	3.0	
Taboose	3.94	12 200	8 000	6 500	3.3	
Goodale	2.29	11 800	7 200	6 500	2.6	Does not include Dry Canyon.
Division	0.95	9 500	7 500	6 500	3.5	
Sawmill	1.32	10 200	7 500	6 500	3.2	
Thibaut (North Fork)	0.53	10 500	7 500	6 500	3.1	
Thibaut (South Fork)	0.07	7 000	6 700	6 500	4.0	Charlies Canyon yields run-off in normal and above normal years.
Oak (North Fork)	3.62	12 600	7 100	6 500	4.2	
Oak (South Fork)	1.03	10 600	7 100	6 500	3.8	
Little Pine	2.02	11 800	7 400	6 500	3.8	Lime Fork yields run-off in normal and above normal years.
Pinyon	2.89	11 500	7 600	6 500	2.7	
Symmes	0.42	9 200	7 400	6 500	3.2	North Fork similar to Lime Fork.
Shepard	0.97	9 900	7 200	6 500	4.5	
Bairs (North Fork)	0.48	9 100	7 100	6 500	5.0	
Bairs (South Fork)	1.21	10 300	7 800	6 500	4.6	
George	2.09	11 200	8 100	6 500	3.5	
Hogback (one-half)	1.08	10 800	7 900	6 500	4.1	
	29.45	....	....	....	....	

The outwash slope (Fig. 2) is the desert portion of the surface of the valley fill, extending from the 6 500-ft. contour at the base of the Sierra Nevada to the upper edge of grass and irrigated land in the valley (3 900 to 4 000 ft.). Its surface is composed of loose boulders, gravel, and sand, deposited during past ages by torrential streams coming from the mountains. This deposit is of

unknown depth, and lies on a buried ancient rocky surface, the higher hills of which appear above the present surface as buttes or knolls. The channels of streams draining the mountain canyons cross this slope in trenches, which, near the mountains, are from 25 to 50 ft. deep.

The valley floor embraces the area between the outwash slope and Owens River, and its surface may be classified as irrigated land, grass or meadow land, alkali land, and desert. The upper edge has a maximum slope of about 120 ft. to the mile, but within a short distance it merges into the practically level valley. The surface is soil to a depth of from 1 to 3 ft., except on the desert land, where it is fine sand. Most of the irrigated land is along the upper margin of the valley floor adjoining the creek channels. The grass or meadow lands lie between and to the east of the ranches, and extend well out into the level valley. The growth is most luxuriant in the spring zone, which is about ½ mile wide and is at the upper edge of the valley floor. Here are numerous small flowing springs, with temperature of about 62°, which start the meadow grass early in the season and keep it green until late in the autumn. Farther out in the valley, the salt grass makes a green carpet from May until late July. In the salt-grass land there is always a deposit of alkali about the plant roots, and the soil surface is crusted. The spring zone, however, is free from alkali. The worst alkali land is practically bare of vegetation and is thickly crusted with white salts. It lies in the more level areas in the center of the valley.

The desert area to the east of Owens River yields no appreciable run-off, and, owing to its light precipitation, it makes no contribution to the ground-water.

The alluvial material which forms the valley fill varies in size from large boulders to fine clay, and, in arrangement, from a thorough mixture of all sizes to layers of well-assorted gravel, sand, and clay. The transporting medium was water, both mountain streams and Owens River taking part in the work. Some of the material was deposited in the beds and on the sides of shifting stream channels, and much of the finer sand and clay was deposited from the quiet waters of a large lake which occupied the lower portion of the valley. The structure of the valley fill, therefore, is complex, and the character of the alluvial material underlying a given locality is difficult to determine without actual examination from borings.

FIG. 2.—EASTERN SLOPE, SIERRA NEVADA.

FIG. 3.—EVAPORATION PAN IN OWENS RIVER.

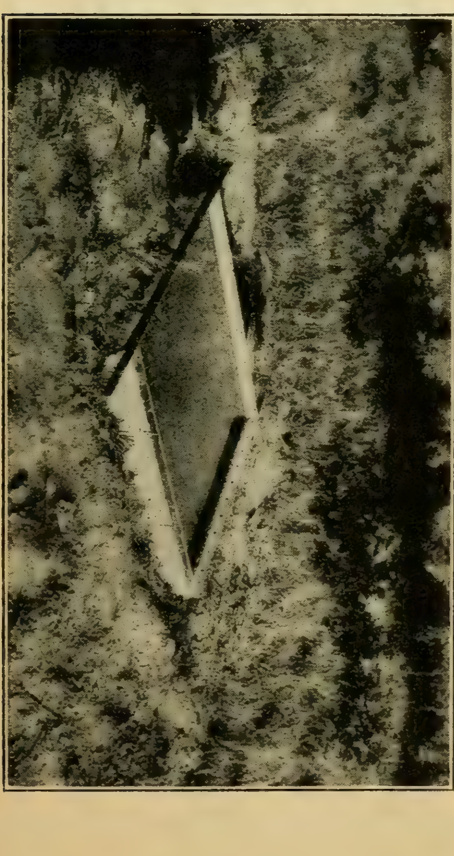
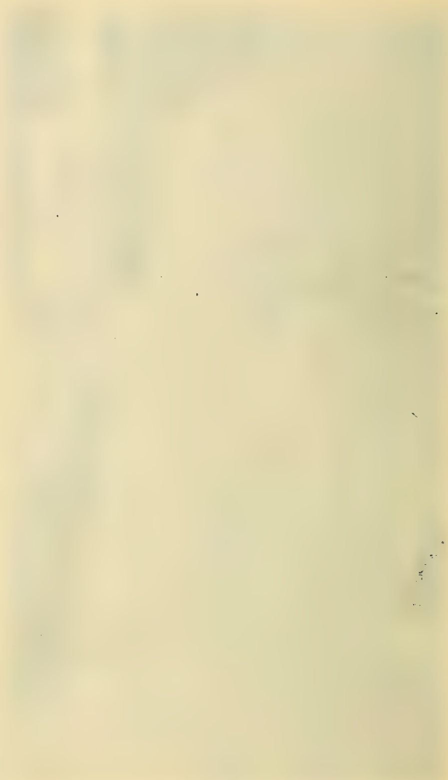


FIG. 4.—EVAPORATION PAN IN SOIL.



A number of borings, ranging in depth from 250 to 500 ft., have been made in the basin by the City of Los Angeles, in connection with the development of the aqueduct supply. In general, the materials encountered were clay, sand, and coarse gravel in layers varying in thickness from a few inches to 150 ft. Coarse material in thin layers interbedded with clay predominates along the upper edge of the valley floor in the spring belt. All wells in this belt yield Artesian flows of from 1 to 2 sec-ft. The material is progressively finer and occurs in thicker strata east of this belt, toward the center of the valley, and the Artesian flows decrease in volume. Near Owens River, fine sand and clay in alternate layers is the only material encountered above the 300-ft. depth.

The streams from the Sierra Nevada were by far the most active in the work of building up the valley fill. Their loads were acquired in the mountain canyons and carried out into the valley, where they were dropped in order of size as the velocity of flow decreased. The old lake level stood at an elevation of about 3 790 ft. for a long period, as shown by beach lines on the east slope of the Alabama Hills. The present 3 790-ft. contour lies near the spring and Artesian belt. The finer materials between the spring belt and Owens River are evidently lake deposits. The ancient lake was contemporaneous with other geologic lakes of the Great Basin, such as Lakes Bonneville and Lahontan. The geologic history of these lakes shows many wide fluctuations of water level, covering long periods of time. The interbedding of fine and coarse material encountered in the spring belt is evidently the result of such fluctuation, as the sudden checking of the velocity of a stream on entering a body of still water results in the immediate deposition of coarse material. The Artesian and spring conditions, therefore, result from hydrostatic pressure on the water entrapped in these wedges of coarse material.

Two cross-sections of the valley, showing the probable geologic structure, were constructed along the Thibaut and Independence Sections (Fig. 5). The topography for these sections was obtained from the U. S. Geological Survey's map of the Mount Whitney quadrangle, and the character of the surface material was determined by field inspection. The exposed slopes of bed-rock on each side of the valley were joined beneath the valley floor, and the arrangement of the material filling the basin thus formed was represented according to

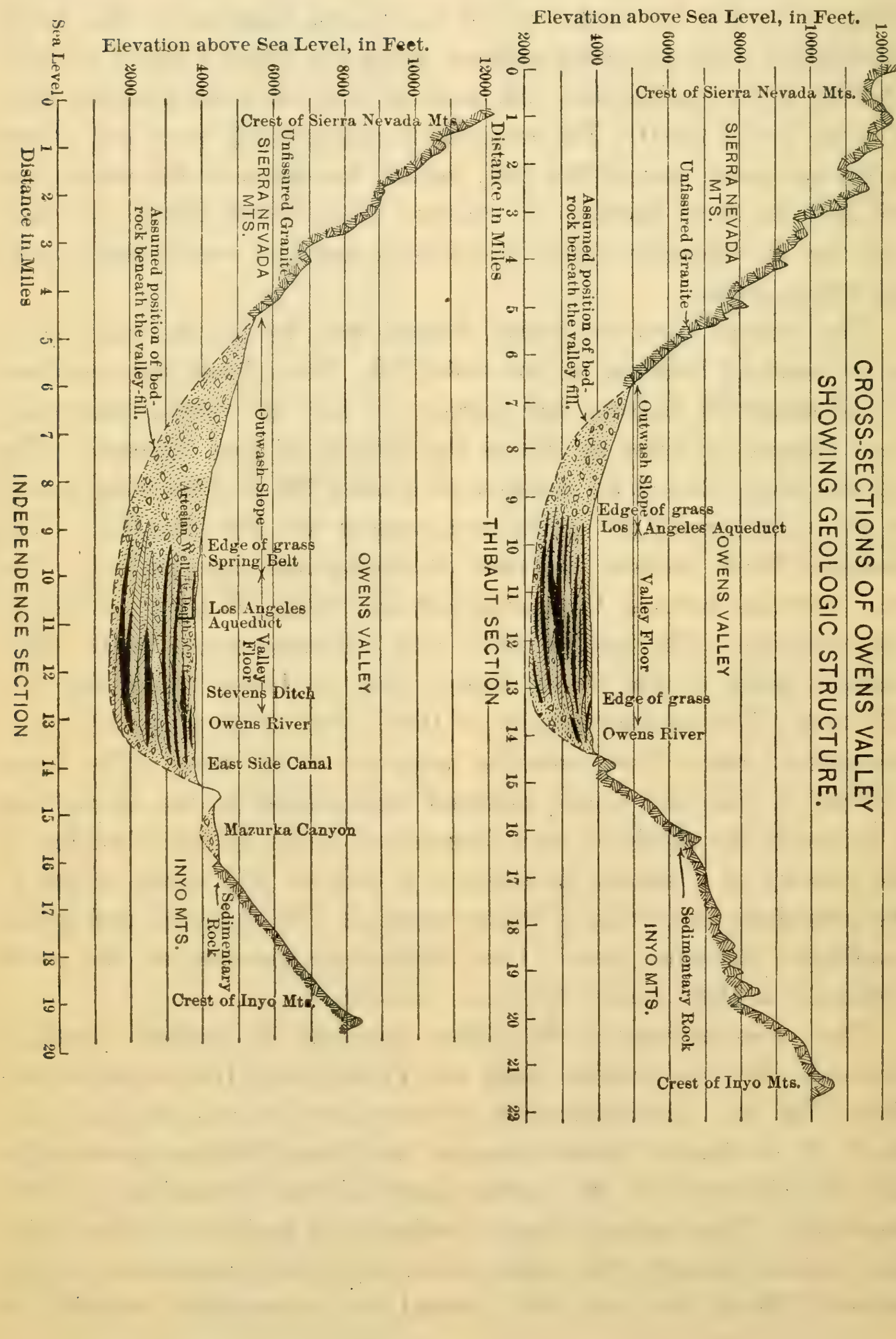

CROSS-SECTIONS OF OWENS VALLEY
SHOWING GEOLOGIC STRUCTURE.
Elevation above Sea Level, in Feet.
12000
10000
8000
6000
4000
2000
Crest of Sierra Nevada Mts.
SIERRA NEVADA MTS.
Unfissured Granite
Assumed position of bed-rock beneath the valley-fill.
Outwash Slope
Edge of grass
Los Angeles Aqueduct
OWENS VALLEY
Valley Floor
Edge of grass
Owens River
Sedimentary Rock
INYO MTS.
Crest of Inyo Mts.
Distance in Miles
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22
THIBAUT SECTION
Elevation above Sea Level, in Feet.
12000
10000
8000
6000
4000
2000
Sea Level
Crest of Sierra Nevada Mts.
Unfissured Granite
SIERRA NEVADA MTS.
Assumed position of bed-rock beneath the valley-fill.
Outwash Slope
Artesian Well
Edge of grass
Spring Belt
Los Angeles Aqueduct
Valley Floor
OWENS VALLEY
Stevens Ditch
Owens River
East Side Canal
Mazurka Canyon
Sedimentary Rock
INYO MTS.
Crest of Inyo Mts.
Distance in Miles
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
INDEPENDENCE SECTION

FIG. 5.

the best available knowledge, the strata of fine material being indicated by solid black. On the diagram, the greatest depth of alluvial filling measures 2 500 ft. in the Independence Section, and 1 800 ft. in the Thibaut Section. Two of the aqueduct wells near the Independence Section reached depths of 500 ft. in alluvial material, and a well drilled by the Southern Pacific Company, opposite the Alabama Hills at Lone Pine Station has reached a depth of 832 ft., entirely in fine sand. There is no reason to suppose that the gravel filling near Independence is less than 2 000 ft. in depth.

The volume of void space, or porosity, of a body of alluvial material of this type is variously estimated by different authorities at from 20 to 35% of the total. Samples of the mixed gravel, sand, and silt of the outwash slopes west of Independence were removed to a depth of 4 ft., without disturbing the natural arrangement of the particles, and weighed dry and after saturation. The results of these tests indicate a porosity of 28% for these samples. The presence of very coarse gravel and boulders in this material would reduce the porosity, and, for the valley fill as a whole, 25% is probably more nearly correct.

Precipitation.

The plan followed in the study of precipitation was to gather and assemble data from which to prepare isohyets for the basin. These appear on Plate II, and are based on the available precipitation data and an intimate knowledge of the local topography and vegetation.

Observations of precipitation were made in the Independence Basin as early as 1865, under the direction of United States Army officers stationed at Fort Independence. The record extends unbroken from September, 1866, to August, 1877, and was obtained under conditions sufficiently similar to permit of combining it with the more recent Weather Bureau record at Independence. The latter covers the periods from September, 1892, to August, 1895, and from September, 1898, to August, 1910, so that there are 26 seasons for which precipitation records are available at Independence. To supplement this record, twenty standard Weather Bureau rain gauges were established, and observations were made during the seasons 1908-09 and 1909-10 (Plate II). These gauges were distributed systematically over the valley floor and outwash slopes, and could all be reached during one day by three mounted observers stationed at points

in the valley where shelter was available. Four records were also available in Owens Valley, outside of the Independence Basin, at Bishop, Lone Pine, Laws, and Keeler. The Bishop and Lone Pine records are kept by co-operative Weather Bureau observers, and cover 15 and 5 years, respectively. The Laws and Keeler records are kept by railroad agents, and are for 13 and 24 years.

The distribution of total precipitation, with respect to geographic location, in the Independence Basin and adjoining areas depends to a great extent on topographic features, notably mountain ranges and valleys, although a consistent variation is also evident with changes in latitude. The controlling topographic feature is the Sierra Nevada, which has a general northwest and southeast trend.

This relation of precipitation and topography is well shown by studying observations made along cross-sections of the Sierra Nevada laid out at right angles to the trend of the range. Two such sections are indicated on Fig. 1 as the Central Pacific and Mokelumne Sections. The relations of mean annual precipitation, altitude, topographic position, and profiles of ground surface are presented graphically for the two sections in Diagrams 1 to 6 of Plate III. The marked similarity in the curves for the two sections indicates that the quantity of precipitation at points in a transverse section of the range conforms to some general law. Elevation, obviously, is not the controlling factor, for above the 5 000-ft. level the precipitation decreases with increase in altitude. The slope of the ground surface appears to be the most important element involved, as is seen from Diagrams 2, 3, 5, and 6 of Plate III. The phenomenon results from the condensation of aqueous vapor due to adiabatic cooling of masses of moist air driven up the slope of a mountain range by the prevailing winds. The region of maximum precipitation is at the lower cloud limit on the windward slope of the range, and above this the latent heat liberated by condensation raises the temperature above the dew point, resulting in decreased precipitation. After crossing the summit of a high range, the descending mass of air contracts in volume, thereby raising the temperature rapidly above the dew point and resulting in marked decrease of precipitation.

The increase of precipitation with elevation was first observed by Mr. S. A. Hill.* in studying rainfall in the northwest Himalayas of

* "California Hydrography," by J. B. Lippincott, M. Am. Soc. C. E., Water Supply Paper No. 81, U. S. Geological Survey, p. 354.

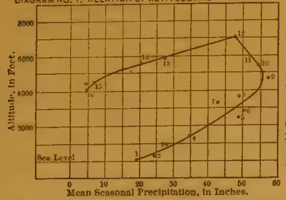
DIAGRAMS SHOWING THE

INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON PRECIPITATION.

AS ILLUSTRATED BY THE SIERRA NEVADA MTS.

CENTRAL PACIFIC GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 1.- RELATION OF ALTITUDE AND PRECIPITATION.



NOTES

Observations on Central Pacific and Mokelumne Groups of gauges by U.S. Weather Bureau.

Observations on Taboose, Oak, and Bairs Groups of gauges by City of Los Angeles. (See Tables 16 and 17.) For location of gauges see Drainage Map of Sierra Nevada Mts. and Drainage Map of Independence Region.

The Season is from Sept. 1st.to Aug. 31st.

DIAGRAM NO. 2.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

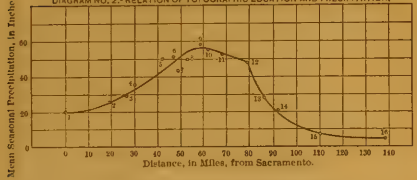
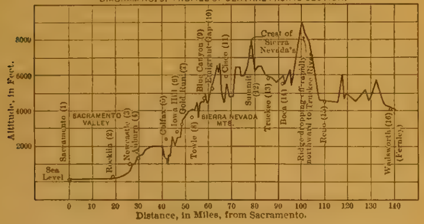


DIAGRAM NO. 3.- PROFILE OF CENTRAL PACIFIC SECTION.



MOKELUMNE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 4.- RELATION OF ALTITUDE AND PRECIPITATION.

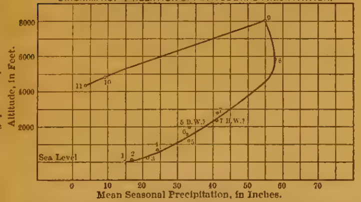


DIAGRAM NO. 5.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

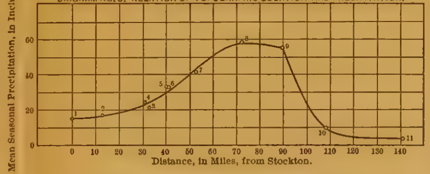
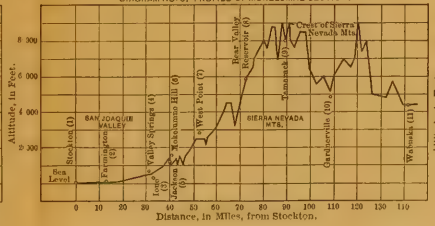


DIAGRAM NO. 6.- PROFILE OF MOKELUMNE SECTION.



TABOOSE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 7.- RELATION OF ALTITUDE AND PRECIPITATION.

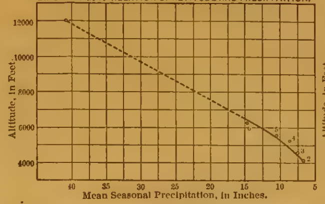


DIAGRAM NO. 8.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

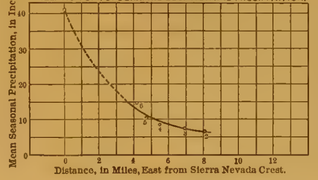
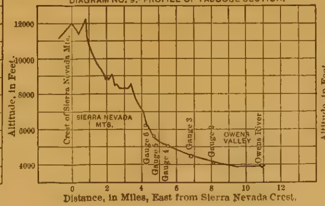


DIAGRAM NO. 9.- PROFILE OF TABOOSE SECTION.



OAK GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 10.- RELATION OF ALTITUDE AND PRECIPITATION.

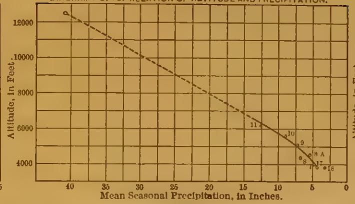


DIAGRAM NO. 11.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

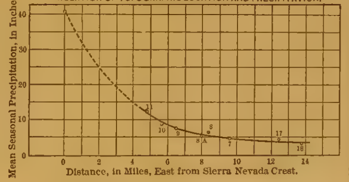
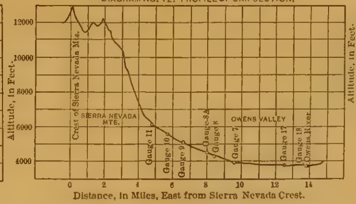


DIAGRAM NO. 12.- PROFILE OF OAK SECTION.



BAIRS GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 13.- RELATION OF ALTITUDE AND PRECIPITATION.

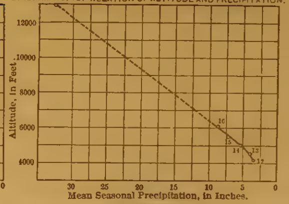


DIAGRAM NO. 14.
RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

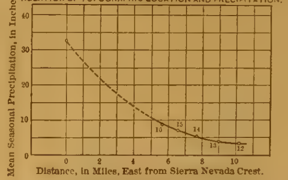
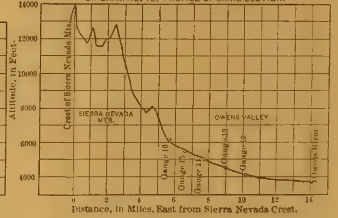


DIAGRAM NO. 15.- PROFILE OF BAIRS SECTION.



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DIAGRAMS S

INFLUENCE OF ALTITUDE AND TOPO

AS ILLUSTRATED BY TH

MOKELUMNE GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 4.- RELATION OF ALTITUDE AND PRECIPITATION.

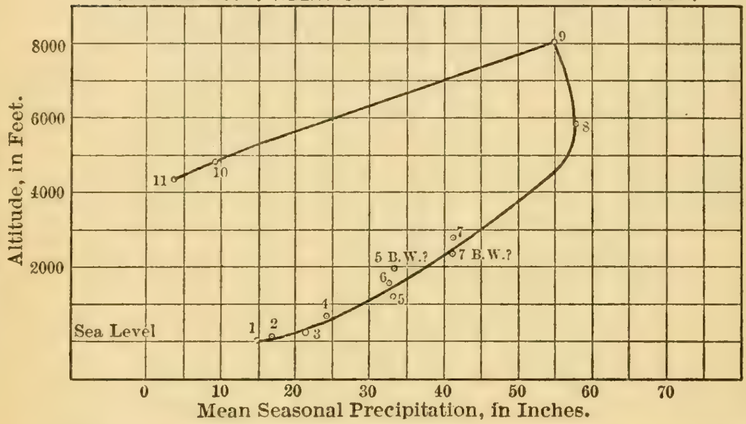


DIAGRAM NO. 5.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

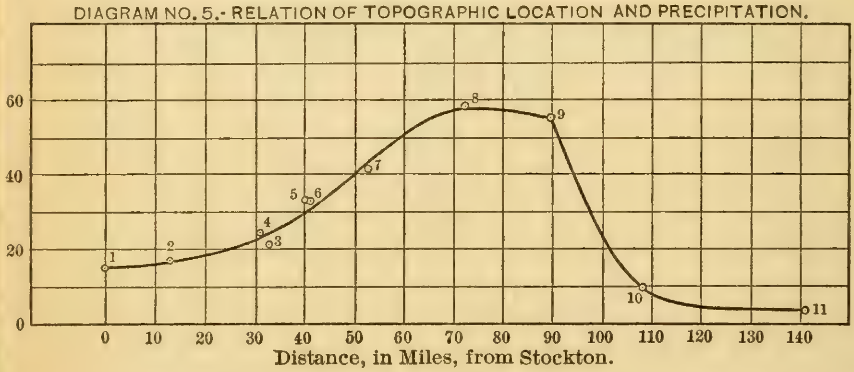
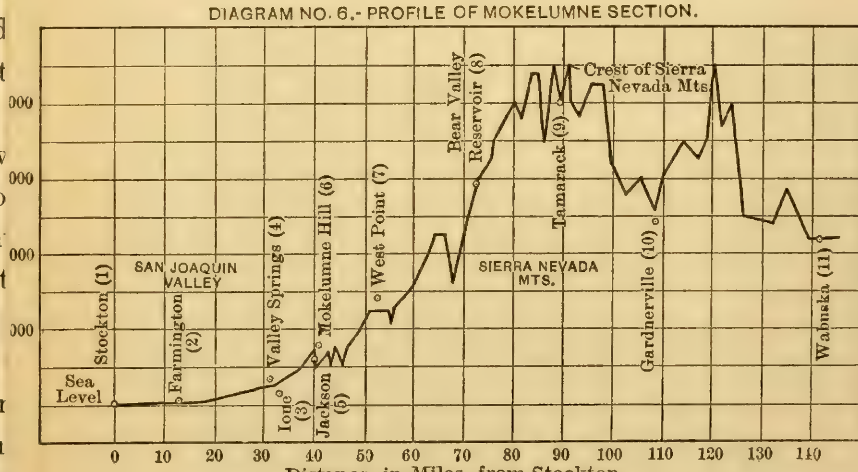
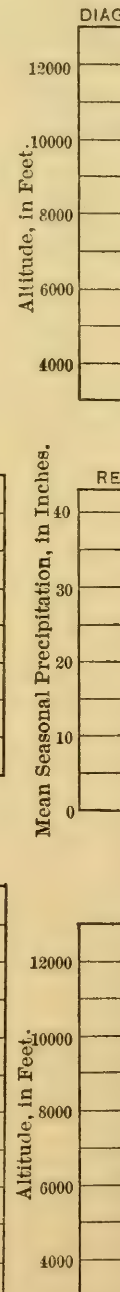


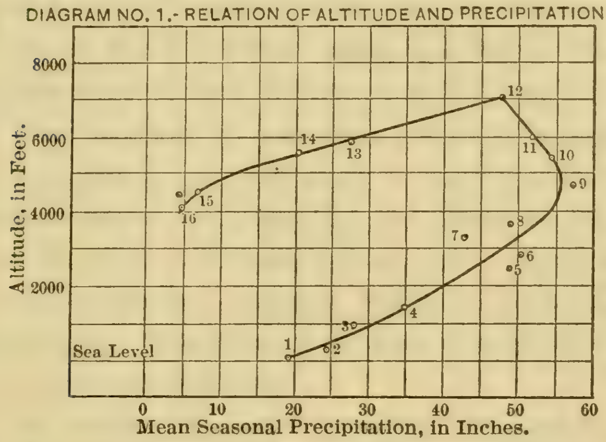
DIAGRAM NO. 6.- PROFILE OF MOKELUMNE SECTION.





CENTRAL PACIFIC GROUP OF PRECIPITATION GAUGES.

DIAGRAM NO. 1.- RELATION OF ALTITUDE AND PRECIPITATION.



NOTES

Observations on Central Pacifi and Mokelumne Groups of gaug by U.S. Weather Bureau.

Observations on Taboose, Oak, and Bairs Groups of gauges by C of Los Angeles. (See Tables 16 an 17.) For location of gauges see Drainage Map of Sierra Nevada M and Drainage Map of Independe Region.

The Season is from Sept. 1st.to Aug. 31st.

DIAGRAM NO. 2.- RELATION OF TOPOGRAPHIC LOCATION AND PRECIPITATION.

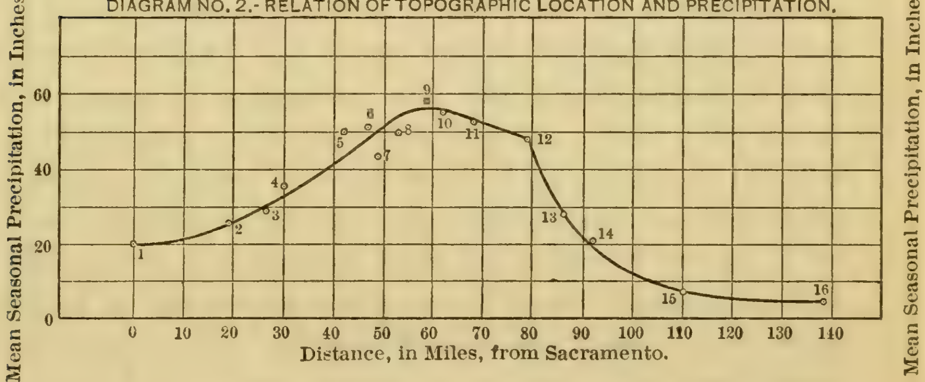
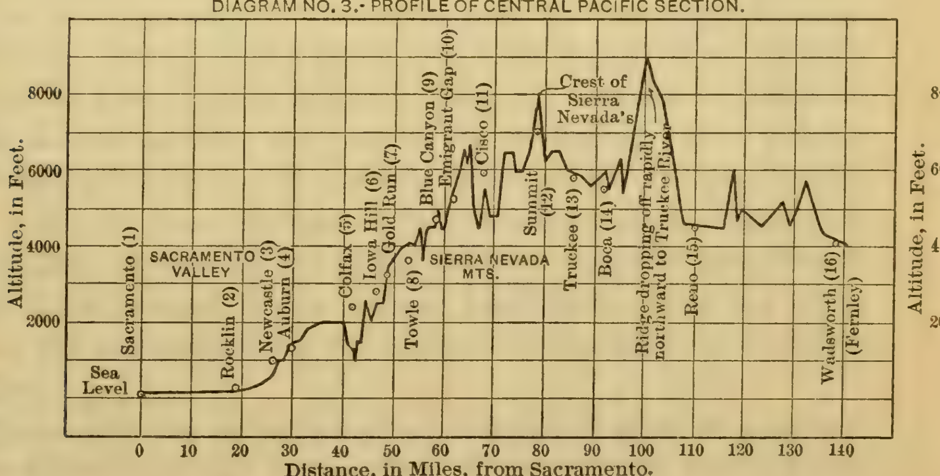


DIAGRAM NO. 3.- PROFILE OF CENTRAL PACIFIC SECTION.



India, and he developed for that region the empirical formula, $R = 1 + 1.92h - 0.40h^2 + 0.02h^3$, in which R represents the quantity of rain and h the relative height, in units of 1 000 ft., above an assumed plane which is itself 1 000 ft. above sea level. This equation, when platted, gives a curve very similar to that shown in Diagrams 1 and 4 of Plate III, the plane of maximum rainfall being 4 160 ft. above sea level. The equation does not apply to conditions on the leeward slope of a range, however, to judge by the discontinuity at the crest line shown on the Sierra Nevada curves. The straight-line relation between precipitation and elevation, which is often assumed in engineering computations, thus appears to have a very limited use, and to be at best a rough approximation.

The Los Angeles Aqueduct precipitation stations in Owens Valley lie in three groups, indicated on Fig. 1 and Plate II, as the Taboose, Oak, and Bairs Sections. The 2-year records for these stations were reduced to averages by comparison with the 26-year record at Independence. The platted curves for these sections (Plate III) are all similar in shape, and agree with the desert slope portion of the Central Pacific and Mokelumne curves. The highest point on each of the Owens Valley curves was obtained from the measured run-off of the canyons crossed by the section and a run-off factor chosen after careful study of precipitation and run-off data for Kings River, which drains the slope of the Sierra Nevada to the west. The precipitation at the mouth of the canyons was known from the rain gauge observations, and the average precipitation over each canyon from the run-off. Most of these canyon drainage areas are isosceles triangles in plan, the base being along the crest of the mountains. Also, judging by the Central Pacific and Mokelumne Sections, the precipitation increases uniformly from the base of the mountains on the desert side to the summit. Hence, the precipitation at the center of area of each drainage basin is equal to the average precipitation over the whole area, and the precipitation at the summit is obtainable by a simple proportion.

Preliminary to the establishment of isohyets or lines of equal annual precipitation for the Independence Basin, a broad study of precipitation was made for the whole Sierra Nevada range from Lake Tahoe to the Mojave Desert (Fig. 1). This was based on the California Water and Forest Association rainfall map of the State, pre-

pared in 1900, as revised between American and Kings Rivers by Edwin Duryea, Jr., M. Am. Soc. C. E., in 1908. With all available rainfall data to date, including the Los Angeles Aqueduct and several private records, the writer has made further revisions over the Southern Sierra and Owens Valley. The Water and Forest Association isohyets, as amended by Mr. Duryea, appear on Fig. 1 as solid lines, and revisions proposed by the writer are represented by dotted lines. Isohyets are shown with greater detail for the Independence Basin on Plate II.

Stream Flow.

Stream flow data essential to the determination of inflow and outflow for an underground reservoir, are the run-off from precipitation, the seepage from or into stream channels, and the flow of springs. Precipitation which finds its way into surface streams without absorption may, along some portion of the channel, percolate into porous gravels and join the subterranean supply. On the other hand, water may escape from the underground reservoir by seepage into stream channels where the general water plane is at a higher elevation than mean water level in the stream. Escape may also occur from springs.

The problem in the Independence Basin was, first, to classify the surface as to run-off characteristics and determine the run-off from each subdivision; second, to ascertain seepage losses from the seventeen tributary mountain streams between the canyon mouths and the valley floor; third, to determine the flow of springs which represent water escaping from the basin; and fourth, to ascertain whether Owens River made or lost water in passing through the basin.

Run-off.—It was early observed that the run-off characteristics of the four areas into which the region was classified for study (Table 1) were similar.

The clay soils of the valley floor occasionally yield a small run-off during and following winter precipitations of 1 in. or more in 24 hours, or warm rain falling on old snow. This water gathers and passes off into Owens River within a few hours by way of four waste channels. A study of the available data shows that the average total run-off from precipitation on the valley floor is about 2 sec-ft. of continuous flow.

The outwash slopes yield no appreciable surface run-off, on account of the porous gravel formation and the great depth to ground-

water. This fact has been established by repeated observations during and after rainstorms and thaws, and is confirmed by the noticeable absence of recent drainage channels or washes, except those of streams which derive their water from high mountain drainage areas.

The intermediate mountain slopes yield a small run-off during May and June, when the temperature at that level is sufficient to melt the accumulated winter snow, but the small streams do not advance far over the outwash slopes before they are entirely absorbed. If the precipitation of the preceding winter is below normal, the snow melts before the hot weather comes, and is absorbed at once. Springs are common along the lower borders of these slopes, the source being the melted snow absorbed by the porous material above and brought to the surface where it comes into contact with impervious formations. In only a few places does such water find its way into living streams.

The high mountain drainage areas have an abundant run-off, and perennial streams flow from all but one of them. The source of this water is precipitation, in the form of snow and rain, which falls within the drainage areas, and to a small extent snow dust carried over the summits by the prevailing west and northwest winds of winter and spring. For all practical purposes, the average discharge at the mouth of the canyon represents the average precipitation within the drainage area minus losses by evaporation from exposed snow surfaces. The underflow from these areas is negligible.

Stream discharge from the canyons is at a minimum from September to April. The flow during these months is remarkably uniform, and is entirely uninfluenced by the current storms, though from 70 to 80% of the annual precipitation occurs between November 1st and March 31st. The low-water flow is derived from springs and from the slow melting of the snow layer exposed to the earth's latent heat. Streams are usually frozen over by November, and as late as April they flow nearly to the mouths of the canyons in tunnels under the snow. Between April 1st and 20th air temperatures increase sufficiently to melt the snow at the lower elevations, and the streams begin to rise. There is an increase in air temperatures and stream flow from this date until the maximum flood crest is reached, some time between June 15th and July 15th, depending on the quantity of snow to be melted. Stream flow then decreases until some time in September,

after which low water prevails. About 70% of the annual run-off of the streams occurs during May, June, July, and August.

Percolation from Stream Channels.—The United States Geological Survey gauging stations on streams draining the high mountain areas are at the lower edges of the outwash slopes, just above the division boxes which apportion the water for use on the ranches of the valley floor. After leaving its canyon each stream traverses several miles of channel before reaching the gauging station, and preliminary observations in June, 1908, showed that considerable water (in some streams 50%) disappeared between the two points. It was necessary, therefore, either to establish regular gauging stations at the canyon mouths and depend on records for short periods, or to devise some means of computing the run-off from the high mountain areas from the existing Government records, which extended over 6 years. The latter method was chosen, and the results have proved very satisfactory.

The loss from these stream channels occurs as percolation into the porous alluvial material, direct evaporation from water surface, and transpiration from vegetation growing along the stream borders. Evaporation and transpiration losses were too small to be detected in current-meter work. As the expense of installing and maintaining weirs was prohibitive, the problem resolved itself into a study of percolation from stream channels.

There are three factors to be considered in a study of the subject: the rate of percolation, the area through which percolation occurs (the wetted perimeter), and the period of time during which a given unit of water is exposed (velocity of flow). The rate of percolation depends on (1) the character of the channel lining and the medium surrounding the channel, as regards size of pores and porosity; (2) the pressure gradient, depending on the difference in level of the surface of the water in the channel and the ground-water surface; and (3) the temperature of the water.

The effect of an increase in the wetted perimeter, other conditions being the same, is obviously to increase the percolation, but such change is accompanied by a proportionally larger increase in the velocity of flow, which reduces the time of exposure of a given volume of water. The net result, considering the total flow, is, therefore, a proportionally smaller percolation, although this effect may be counter-

acted to a certain extent by the scouring of a non-porous channel lining due to the increased carrying power of the stream. The whole matter is affected by so many indeterminate conditions that a general mathematical analysis is impossible, but, with these ideas in view, a study was made of each channel within the ordinary range of temperature and discharge.

The field work consisted of making comparative current-meter measurements at upper and lower stations on each creek, giving proper allowance of time for the passage of water between the two points. The measurements were made at intervals of from 6 weeks to 2 months, and extended over the period from June 15th, 1908, to September 15th, 1909, including the high-water periods of wet and dry seasons. Gauging sections were prepared at the mouth of the canyon on each creek. Estimates of the time required for the passage of water between stations were based on actual trial with aniline dye. Very little fluctuation in discharge was observed in any of the creeks between 8 A. M. and 5 P. M., even in the high-water period.

The temperature of the water as it issues from the canyons varies from 35° to 42° Fahr., in winter, and from 48° to 53° Fahr., in summer. In winter the temperature does not increase much as the water travels toward the valley. After leaving the protecting cover of the snow in the canyons during December and January the water actually becomes colder, ice prevailing for several weeks. In summer there is an average increase of 10° between the stations at the mouth of the canyon and in the valley.

Several methods were attempted for generalizing the results, but the most satisfactory was a graphical one, in which losses were platted as abscissas and stream discharges as ordinates with rectangular coordinates (Plate IV).

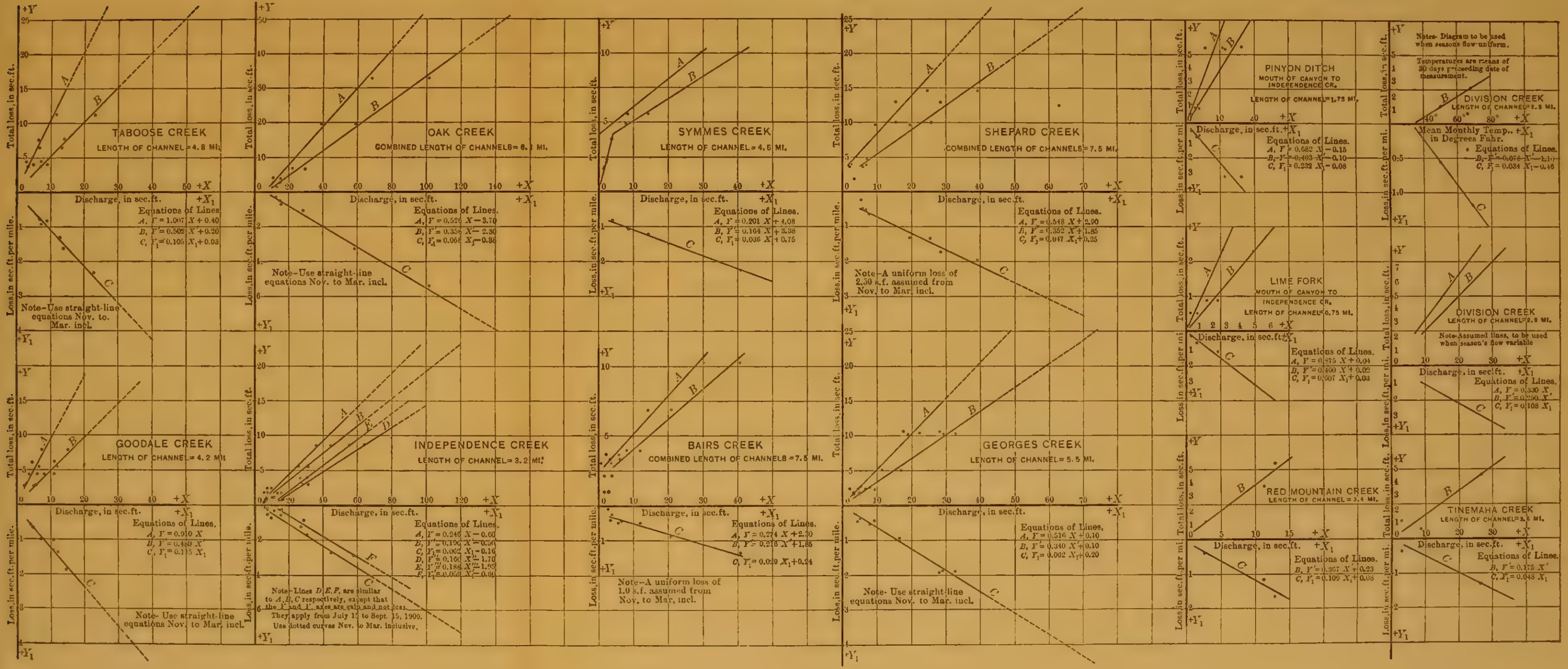
It was found that for each channel a straight line expressed the relation of these two quantities from April to October, inclusive. During the remaining 5 months, the relation is not clear, but the total loss is then so small that it can be obtained by inspection without affecting the accuracy of the computed discharge at the mouth of the canyon. Total losses are platted on the basis of discharge, both at lower and upper stations, so that, in correcting the Government records to obtain the true yield from high mountain drainage areas, the quantity desired can be obtained at once by entering on the X-axis

the discharge at the lower or Government station. It is also possible at the same time to read the loss, in second-feet to the mile, from the straight line below the X-axis.

The diagrams on Plate IV were prepared for the purpose of computing (1) seepage losses above the U. S. Geological Survey gauging stations, and (2) the actual yield of high mountain drainage areas. In obtaining the field data, discharge measurements were made with small Price current meters during the period, June to August, 1909, inclusive, covering a season of small run-off and one of large run-off. The accuracy is up to the standard for a stream of this type. Measurements of medium and high stages were difficult, on account of the rough sections and very high velocities. The upper gauging station is at the mouth of the canyon, or below impervious dikes forcing seepage water to the surface. The average elevation is 6 000 ft. The lower station is that used by the U. S. Geological Survey (unless otherwise noted), which is above the diversion and has an average elevation of 4 100 ft. Below the latter there is ½ mile or more of channel suffering a large seepage loss which is not included. The length of the channel was obtained by scaling from the Mt. Whitney Quadrangle, and, for Taboose, Red Mountain, and Tinemaha Creeks, from a triangulation survey. Elevations were obtained in a similar manner. In using the diagrams, the straight-line relation applies from April to October, inclusive, during which time temperature and discharge vary similarly. The dotted portions are not supported by field data, and are to be used with judgment. From November to March, inclusive, temperature is the only effective variable, and arbitrary values for loss have been selected. Line "A" expresses the relation of total loss to discharge at the U. S. Geological Survey Station; Line "B" expresses the same relation at the mouth of the canyon; and Line "C" expresses the loss per mile to discharge at the mouth of the canyon. The diagrams are arranged so that all the four values involved may be obtained by entering any one of them.

There are some interesting conclusions to be drawn from these diagrams. In general, the quantity of water percolating from the channels studied varies with the time of year and with different channel conditions. Variation with the time of year is due to the combined effects of temperature, area of wetted perimeter, and velocity of flow. These work more or less in harmony during April to October, inclusive,

SEEPAGE LOSS DIAGRAMS FOR OUTWASH SLOPE CHANNELS OF STREAMS DRAINING HIGH MOUNTAIN AREAS IN THE VICINITY OF INDEPENDENCE



and produce the straight-line relation of total loss and discharge. From November to March, inclusive, canyon discharges remain practically constant, showing that variations are largely controlled by temperature. Discharges are then so small, however, that errors of measurement are appreciable, and losses by evaporation have greater weight, so that the true relation of loss and temperature does not appear. A possible relation between total loss and temperature is suggested by the results on Division Creek, where the discharge at the mouth of the canyon was practically uniform during the period of study. Using as ordinates the mean air temperature at Independence for the 30 days preceding each date of measurement, we obtain a straight line which crosses the X-axis at about 35 degrees. A line supported by additional data might cross nearer 32°, the temperature at which percolation becomes physically impossible.

The character of the surrounding medium was the only channel condition which noticeably affected percolation. The loss from a channel crossing fissured lava, even where the lava was covered by a thin sheet of alluvium, was 30% greater than that in coarse alluvium. The streams studied do not overflow their channels, so that the effect of varying channel slopes and wetted perimeter could not be studied.

The run-off from each mountain canyon was computed from U. S. Geological Survey monthly mean discharge records, by use of the diagrams. The results are summarized in Tables 4 and 5. The missing seasons were estimated from the unbroken records of neighboring streams after a detailed study of yield per square mile and annual departure from normal for each stream. The long-term mean discharge was obtained by comparison with the Kings River record, which covered 21 years. This stream was chosen because its drainage area adjoined most of the Owens Valley streams and because conditions affecting run-off were more nearly similar than on any other stream. The results indicate that the total annual mountain run-off during the period of record was 153 annual sec-ft. and the normal 130 annual sec-ft. This total does not include Red Mountain and Tinemaha Creeks, which pass out of the basin after crossing a portion of the outwash slope. The normal run-off per square mile for streams north of the Kings-Kern divide with 60% or more of area above 10 000 ft., is 1.75 sec-ft., and, for streams similarly situated, with less than 60% above 10 000 ft., 1.18. For streams south of the Kings-Kern divide,

TABLE 4.—SEASONAL DISCHARGE, IN SECOND-FEET, AT UNITED STATES GEOLOGICAL SURVEY STATIONS, OF CREEKS TRIBUTARY TO INDEPENDENCE REGION.

(Figures in parentheses are estimated.)

Creek.	YEAR BEGINNING SEPTEMBER 1ST.						Observed 5-year mean.	Computed 21-year mean.
	1904.	1905.	1906.	1907.	1908.	1909.		
Taboose	(5.7)	11.4	10.3	4.7	8.6	5.9	8.2	6.5
Goodale	(3.9)	5.4	6.6	3.8	6.4	5.3	5.5	4.3
Dry Canyon	0	0	0	0.0	0	0	0	0
Division	(3.9)	7.2	10.9	7.6	9.7	9.9	9.1	7.2
Sawmill	....	5.4	(7.6)	(5.0)	7.3	7.2	6.5	5.1
Thibaut	(0.5)	(1.0)	(1.0)	0.8	0.9	0.2	0.6	0.5
Oak	(13.0)	31.8	23.9	15.8	30.8	18.4	24.1	19.0
Little Pine, Pinyon	(10.7)	28.5	22.5	11.8	25.8	17.2	21.2	16.7
Symmes	....	(2.8)	3.1	0.8	6.3	1.1	2.8	2.2
Shepard	....	23.1	11.0	7.2	12.9	7.7	12.5	9.8
Bairs	....	8.0	4.8	2.0	6.1	2.4	4.7	3.7
George	....	18.6	10.9	6.5	13.0	6.8	11.2	8.8
Hogback	(0)	(1.0)	(0.5)	(0)	(0.5)	0	0.4	0.3
	....	144.2	113.1	66.0	128.3	82.1	106.7	84.0
Percentage of totals at mouth of canyon....	....	68	63	61	66	63	65	65

TABLE 5.—SEASONAL DISCHARGE, IN SECOND-FEET, AT MOUTH OF CANYON, OF CREEKS TRIBUTARY TO INDEPENDENCE REGION.

(Figures in parentheses are estimated.)

Creek.	YEAR BEGINNING SEPTEMBER 1ST.						Observed 6-year mean.	Computed 21-year mean.
	1904.	1905.	1906.	1907.	1908.	1909.		
Taboose	11.9	19.2	20.4	9.8	16.0	12.4	15.0	12.7
Goodale	7.6	9.9	12.3	7.2	11.2	10.2	9.8	8.3
Dry Canyon	(3.9)	(5.1)	(6.3)	(3.7)	(5.7)	(5.2)	5.0	4.2
Division	(4.4)	6.9	11.2	6.9	8.7	9.5	7.9	6.7
Sawmill	(4.0)	6.5	9.1	5.5	7.3	7.2	6.6	5.6
Thibaut	(1.9)	(3.1)	(4.4)	(2.7)	(3.5)	(3.4)	3.2	2.7
Oak	16.7	43.2	33.8	21.2	42.6	25.0	30.4	25.8
Little Pine	10.7	24.6	20.5	12.0	24.3	16.7	18.1	15.3
Pinyon	(3.6)	(8.9)	(6.6)	(4.2)	9.2	3.5	6.0	4.8
Symmes	(3.5)	(8.8)	7.0	3.7	10.2	4.9	6.2	5.3
Shepard	(11.1)	31.4	18.9	13.5	2.09	14.1	18.3	15.5
Bairs	(4.3)	11.8	7.7	4.2	9.4	4.8	7.0	5.9
George	(8.2)	23.8	16.5	9.8	18.7	10.3	14.6	12.4
Hogback	(2.8)	(7.8)	(5.7)	(4.2)	(6.8)	(3.7)	5.2	4.4
	94.6	211.0	180.4	108.6	194.5	130.9	153.3	129.6
Percentage reaching U. S. G. S. Stations.	....	68	63	61	66	63	65	65

the normal run-off is 1.36 and 0.86 sec-ft., respectively. It is also of interest to note that only 65% of this run-off reaches the Government gauging stations.

Springs.—The occurrence of springs in the basin is due to the reappearance of water which originally fell within its boundaries as precipitation and was absorbed. There are, in general, three types of springs which give rise to surface streams: those which derive their supply from precipitation on the intermediate mountain slopes, and appear at the base of these slopes; those which derive their supply from precipitation and stream percolation, and appear along the upper edge of the grass land; those which derive their supply from precipitation on lava flows, and appear at the lower borders of the flows.

The springs of the first type are not deep seated; they represent the drainage from the superficial deposits lying on the triangular mountain slopes between canyons. The temperature of their water is about 47° or 48° Fahr., and the flow in many of them increases in early summer and decreases during late summer and autumn. The water from most of these springs sinks into the porous gravels of the outwash slope, and joins the main body of ground-water in the basins.

The line of springs along the upper edge of the grass land represents the intersection of the natural surface of the ground and the surface of the ground-water. The water has penetrated rather deeply into the gravel fill, and issues with a temperature of about 62° Fahr., which is 5° higher than the mean annual temperature at Independence and 1° lower than that of water flowing from Artesian wells in the same location. The flow of these springs is variable, being least in late summer and greatest in early spring, with regular fluctuation between these dates, evidently depending on ground-water stages within the grass area. Only during the winter months is the discharge sufficient to be the source of surface streams which flow any considerable distance, and even then there are only a few of such streams which reach Owens River. Most of the yield of these springs is lost by evaporation and transpiration. The winter discharge of individual springs varies from 0.5 sec-ft. down to a quantity which is only enough to fill small pools of standing water, from which the evaporation equals the yield. The total winter discharge from all these springs is about 4 sec-ft.

The springs issuing from the lava formations are unique in having uniform discharges throughout the year and a temperature of 57° Fahr. The water is probably derived from precipitation on the lava surface, absorbed by the porous rock, and, by reason of the peculiar formation, gathered and delivered at the lower margin of the flow. The largest of these is Blackrock Springs (Plate II), 9 miles north of Independence. It has a discharge of 23 sec-ft., which flows out across the valley floor in two sloughs, each emptying into a series of shallow lakes. From November to March, inclusive, an average flow of about 7 sec-ft. reaches Owens River, but during the remainder of the year all the water is lost by seepage, evaporation, and transpiration. Hines Spring is 3 miles north of this spring, and has a continuous yield of about 4 sec-ft. Approximately, 1 sec-ft. finds its way into Owens River during the winter, but is lost during the remainder of the year. Campbell Spring is east of Owens River, 1 mile north of Aberdeen. It has a yield of about 0.5 sec-ft., and discharges directly into the river. Upper and Lower Seeley Springs are just above and just below Charlies Butte, and discharge directly into Owens River. The upper spring has a flow of 9.5 sec-ft., which is included in measurements of Owens River at the Butte. The lower one has a flow of 1.5 sec-ft.

Owens River.—Owens River flows lengthwise of the Independence Basin for 29 miles, although the actual length of its channel is possibly 20% greater, owing to its sinuosity. It is the drainage outlet for the waste surface water of the region, including the run-off from the valley floor, the yield of springs, and a small portion of the run-off from high mountain drainage areas. In order to account for all escaping surface waters, and determine the condition of the river channel with regard to seepage, observations of river discharge were made daily near the north and south boundaries of the region, and measurements of discharge into and diversion from the river channel were made between these two points. Complete data are available for 1909 and 1910. Analysis of these data shows that seepage losses occur during high-water, and seepage gains during low-water stages. The net result is a loss between Charlies Butte and Whitney Bridge, which can be accounted for by channel evaporation. The water plane of the valley on each side of the river lies between high- and low-water levels in the river. Hence, seepage gain and loss are the result of local ab-

sorption and drainage along the river channel, and have no relation to the general ground-water situation of the basin.

Evaporation and Transpiration.

Evaporation from Water Surfaces.—Measurements of evaporation from free water surfaces were made under three conditions: from a pan floating in a body of water, from a pan placed in the soil, and from a deep tank placed in the soil. The first and second were designed to furnish data regarding evaporation from reservoir surfaces and from areas of shallow flood water, respectively. The third was desired for purposes of comparison with records of evaporation from soil. The pans, which were of the pattern used by the U. S. Reclamation Service, were 3 ft. square and 10 in. deep, and were of galvanized sheet-iron. Observations were made by replacing the quantity evaporated with a cup having a capacity equal to a depth of 0.01 in. in the pan. The initial height of the water surface was such that a pin, projecting from the center of the pan and remaining at a fixed height, 2 in. below the rim, was just submerged. The deep tank was circular, 3½ ft. in diameter and 4 ft. deep, and observations were made in a stilling well with a hook-gauge and vernier scale reading to 0.01 in. The records were all kept near Independence, and observations were made every second day in summer and every fourth day in winter.

The record for the pan in water (Table 6) is available from August 4th, 1908, to June 1st, 1911. The pan, Fig. 3, at first, was in Blackrock Slough, but was moved to its final location, in Owens River at Citrus Bridge, on May 7th, 1909. The pan was supported by a timber float, which protected it from splashing water. The depth of water beneath the pan varied from 1 to 5 ft., depending on the river stage. The river water had a moderate velocity, and varied in temperature from about 75° Fahr., in summer, to about 40° Fahr., in winter. The river banks averaged 4 ft. high above the water surface, and the pan was about 30 ft. from them. Rain gauge No. 18 was 100 ft. away, on the river bank, and was observed in connection with the evaporation record.

The record for the pan in soil is broken. It extends from August 1st, 1909, to November 30th, 1909, and from March 14th, 1910, to June 1st, 1911. The pan, Fig. 4, was in the valley floor at the soil evaporation experiment station, about 3 miles east of Independence.

It was set in a shallow excavation with soil banked up to about half the depth of the pan. Water temperatures range from 95° Fahr., in summer, to 32° Fahr., in winter. The surface temperature was about 1° warmer than that for the mixed contents of the pan. Table 7 summarizes the results by months for this pan, and, by comparing it month by month with the evaporation from the pan in water, an average excess of about 33% is observed. This is probably due to the higher temperature of the water in the pan in soil during the hours of sunlight.

TABLE 6.—DEPTH OF EVAPORATION, IN INCHES, FROM WATER SURFACE NEAR INDEPENDENCE (PAN IN WATER).

Month.	1908.		1909.		1910.		1911.		Average percentage of annual evaporation.
	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	Total.	Rate per 24 hours.	
January	....	....	1.60	0.052	1.75	0.056	1.65	0.053	2
February	....	....	2.40	0.086	2.50	0.089	2.35	0.084	4
March	....	....	4.70	0.152	5.15	0.166	3.70	0.119	7
April	....	....	7.30	0.243	7.05	0.235	6.25	0.208	11
May	....	....	9.60	0.310	8.29	0.267	8.01	0.258	13
June	....	....	10.10	0.337	9.90	0.330	....	....	15
July	....	....	10.40	0.335	8.50	0.274	....	....	14
August	* 4.90	0.222	8.00	0.258	8.20	0.264	....	....	12
September	5.30	0.176	6.60	0.220	6.30	0.210	....	....	10
October	3.50	0.113	3.90	0.126	4.20	0.135	....	....	6
November	2.50	0.083	2.60	0.087	2.36	0.079	....	....	4
December	1.50	0.048	(1.85)	0.060	1.24	0.040	....	....	2
	....	....	69.05	0.189	65.44	0.179	....	....	100

* August 10th to 31st, inclusive.

The deep-tank record extends unbroken from April 16th, 1909, to Dec. 31st, 1911. The tank, Fig. 6, was at the soil evaporation experiment station, and was set in the soil with the upper rim flush with the surface. The water surface was not allowed to fall more than 4 in. below the rim. The temperature of the surface water varied from 80° Fahr., in the heat of summer to freezing in winter. Except during freezing weather, the average temperature of the contents of the tank was 5° less than that of the surface layer. The presence of the surrounding soil makes the range in temperature less than that for the shallow pan. The record, which is presented in Table 8, indicates an annual depth of evaporation practically equal to that from the pan in water at Citrus Bridge. The monthly distribution is more uniform,

TABLE 7.—DEPTH OF EVAPORATION, IN INCHES, FROM WATER SURFACE NEAR INDEPENDENCE (PAN IN SOIL).

Month.	1909.			1910.			1911.		
	Total.	Rate per 24 hours.	Percentage of evaporation from pan in water.	Total.	Rate per 24 hours.	Percentage of evaporation from pan in water.	Total.	Rate per 24 hours.	Percentage of evaporation from pan in water.
January	....	....	....	....	....	....	2.25	0.073	138
February	....	....	....	....	....	....	2.25	0.080	95
March	....	....	....	* 4.25	0.236	....	4.80	0.155	130
April	....	....	....	9.50	0.316	135	8.12	0.271	130
May	....	....	....	10.61	0.342	128	10.25	0.330	128
June	....	....	....	11.95	0.398	121	....	....	....
July	....	....	....	12.55	0.405	148	....	....	....
August	10.70	0.345	134	11.80	0.381	144	....	....	....
September	8.50	0.283	129	8.80	0.293	140	....	....	....
October	5.80	0.187	149	5.60	0.180	133	....	....	....
November	3.80	0.127	146	2.85	0.095	121	....	....	....
December	....	....	....	1.60	0.052	129	....	....	....
	....	....	....	....	....	133	....	....	....

* March 14th to 31st, inclusive.

TABLE 8.—DEPTH OF EVAPORATION FROM WATER SURFACE NEAR INDEPENDENCE.

DEEP TANK IN SOIL.

Month.	1909.			1910.			1911.			Percentage of annual evaporation during each month.
	Total, in inches.	Rate, in inches per 24 hours.	Percentage of evaporation from pan in water.	Total, in inches.	Rate, in inches per 24 hours,	Percentage of evaporation from pan in water.	Total, in inches.	Rate, in inches per 24 hours.	Percentage of evaporation from pan in water.	
Jan.	....	....	....	2.00	0.064	114	2.30	0.074	139	3
Feb.	....	....	....	2.90	0.104	116	2.55	0.091	108	4
Mar	....	....	....	5.60	0.180	109	3.95	0.127	107	7
Apr	2.90*	0.193	....	7.40	0.246	105	6.80	0.226	84	10
May	7.50	0.242	78	7.71	0.248	93	7.90	0.254	77	12
June	7.80	0.260	77	8.60	0.287	87	6.65	0.222	....	11
July	7.90	0.254	76	8.30	0.268	98	8.60	0.277	....	12
Aug	8.20	0.264	102	8.80	0.284	107	9.65	0.311	....	14
Sept	7.20	0.240	109	7.30	0.243	116	7.16	0.239	....	11
Oct	5.00	0.161	128	5.15	0.166	123	4.90	0.158	....	7
Nov	3.30	0.110	127	3.10	0.103	131	3.00	0.100	....	5
Dec	(2.20)	0.071	119	2.15	0.069	173	(2.50)	0.081	....	4
Totals..	......	......	......	69.01	0.188	106	65.96	0.180	....	100

* For period, April 16th to 30th, inclusive.

there being 70% of the total during the 6 summer months and a difference of 27 in. between summer and winter evaporation. The effect on evaporation of the modified temperature extremes of the soil is well shown by comparison with the record for the pan in water (Table 6). The temperature conditions for the deep tank agree closely with those of the surrounding soil.

Evaporation from Ground Surface.—Water in the surface layers of the ground is subject to evaporation, either directly from the soil or through vegetation by the process of transpiration. It is available for evaporation in Owens Valley under two conditions: temporarily, following a rainstorm or sudden thaw, and permanently, within areas where the average depth to ground-water does not exceed 8 ft. The total evaporation under the first condition is relatively unimportant, because of the infrequency of storms and the small quantity of precipitation, and no attempt was made to measure it. Under the second condition, however, evaporation losses are large, for, not only is soil capillarity able to draw gravity water to the surface, but roots of vegetation, such as wild grass, penetrate the soil to ground-water and become the channels by which a large quantity of moisture is conveyed into the atmosphere. Evaporation from bare soil combined with transpiration is, in fact, the most important element entering into computations relating to ground-water for this region. So few data are available on the subject that extended observations were undertaken.

Owens Valley is an ideal location for carrying on such experiments. In the first place, the source of water available for evaporation may be kept under the complete control of the observer as regards the quantity and rate of supply. Storms are rare, and the total precipitation is small, so that little uncertainty exists from this cause regarding the quantity of percolation from precipitation on the surface of a body of isolated soil. Second, the method by which the surface soils of the valley floor are kept moist can be reproduced artificially on a small scale with only a slight departure from natural conditions. The source of supply for soil moisture is a permanent ground-water surface from which water is drawn by capillary forces. This ground-water is replenished by percolation from the precipitation and surface water of the intermediate mountain and outwash slopes, which seeps laterally toward the valley floor and lies beneath it under hydrostatic pressure sufficient to maintain a permanent ground-water surface.

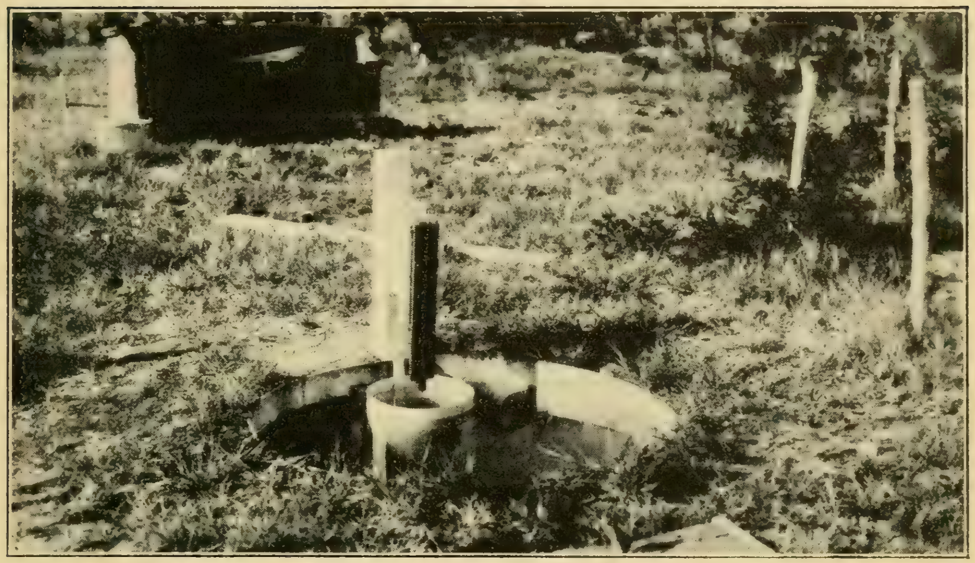


FIG. 6.—DEEP-WATER EVAPORATION TANK.

FIG. 7.—SOIL TANK No. 3 IN OPERATION.

Similar pressure can be reproduced in the bottom layer of an isolated body of soil, and capillary forces can be depended on to raise moisture to the surface. Finally, the large annual depth of evaporation makes possible a more accurate determination of its quantity than in a less arid region. Experiments carried on under these conditions have been very satisfactory.

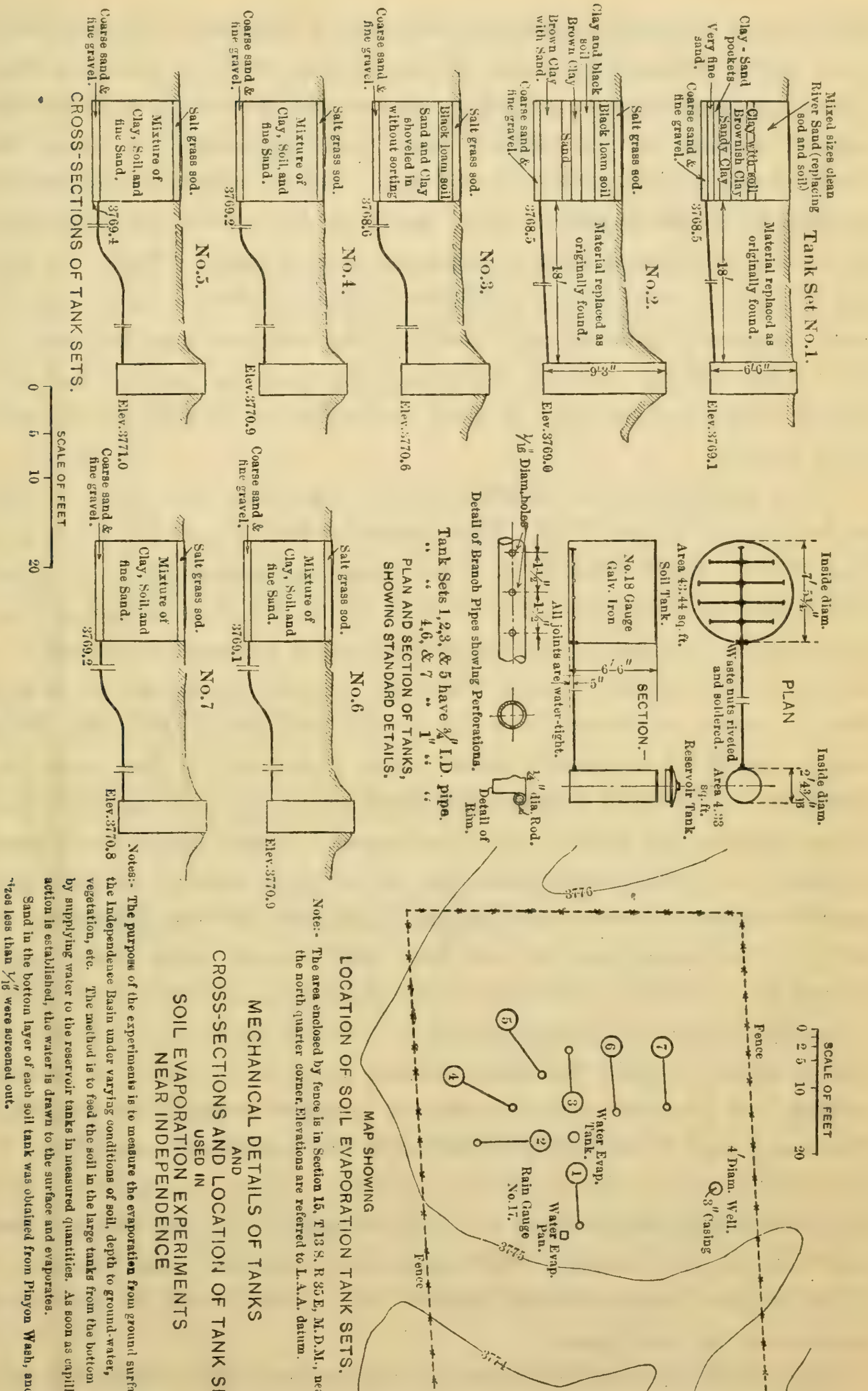
The rate of evaporation from soil depends on the temperature of the air and soil, the quantity of moisture already in the immediately surrounding atmosphere, the quantity of moisture in the surface layers of the soil, and the character of the vegetation and other soil covering. The first two of these factors have the same effect on soil evaporation as on that from a free water surface—higher air and soil temperatures result in increased evaporation, as does also dryer atmosphere or increased movement of wind. The third factor is directly proportional to the rate of evaporation, because the loss of moisture occurs from soil grains at or very near the surface. The quantity of moisture in the soil available for evaporation thus depends on the character of the soil, as regards capillarity and depth to the ground-water surface. For example, in a coarse, sandy soil, "gravity water" will be drawn to the surface through the capillary spaces from depths not exceeding 4 ft., and, in a fine sandy or clayey soil, water will be drawn from depths as great as 8 ft. The last factor, the extent and character of vegetation, affects the evaporation rate both through the activity of transpiration and the effect on capillarity. Plant roots are continually absorbing water from the soil; this water passes off into the atmosphere through the leaves, and the evaporation losses from soil are greatly increased thereby. The roots of native salt grass will penetrate to a depth of 8 ft. in search of water. A further effect of the growth of vegetation is to increase the vertical capillary flow of moisture through soil by way of the many tubes filled with the rotted fiber of dead roots. These tubes are the result of years of growth, and penetrate the soil in all directions above the ground-water surface.

The purpose of the experiments was to obtain data sufficiently complete to compute the total volume of water annually lost by evaporation and transpiration from the valley floor. This involved making observations under the various local conditions which affect soil evaporation. The plan was to reproduce natural conditions in isolated bodies of typical soil and determine the evaporation therefrom for

varying climatic conditions, depths to ground-water, soils, and vegetation.

The experimental equipment consists of two galvanized-iron tanks, 6½ ft. in depth, connected at the bottom by an 18-ft. length of galvanized pipe. (See Fig. 8.) The smaller tank is 2 ft. $4\frac{3}{16}$ in. in diameter, and has a tight-fitting cover. The larger tank is 7 ft. 5¼ in. in diameter, and has a system of branching perforated pipes at the bottom connected with the pipe from the smaller tank. The two tanks and all connections are water-tight, and water poured into the smaller or reservoir tank passes into the larger or soil tank and escapes through the perforations. These two tanks were placed in excavations of proper size to receive them, the soil tank was filled with the excavated soil, and the reservoir tank was filled with water. A 6-in. layer of screened gravel, too coarse to enter the $\frac{1}{16}$-in. perforations, was laid in the bottom of the soil tank in order to insure an uninterrupted and well-distributed feeding of water from the reservoir tank into the superimposed soil. As soon as the material became saturated and capillary action was established to the surface, the water level in the soil was brought to the desired depth and kept there by supplying water to the reservoir tank in measured quantities. Volumetric measurements of water poured into or withdrawn from the reservoir tanks were made with an ordinary gallon measure. Accumulation or depletion of the supply in the reservoir tank was determined volumetrically by measuring the depth of water with a steel tape. The volume passing out of the reservoir tank during a given period represents the total evaporation from the soil tank during that period.

The position of the ground-water surface in the soil tank was determined by measuring its depth below the ground surface in 2-in. augur holes bored in the soil to a proper depth. Measurements were made from a fixed point with a steel tape weighted at the end and chalked before each observation. Three holes were placed in each tank, half way between the center and rim, on radii 120° apart. The holes were not bored deep enough to reach the bottom layer of coarse gravel, and the water level in them represented the ground-water surface in the surrounding soil. An average of the observations made at a given time was assumed to represent the general depth to ground-water for the tank at that time. The tendency of the sides of the holes to cave in and the bottom to fill with sand was controlled by casing



Tank Set No.1.
Mixed sizes clean River Sand (replacing sod and soil)
Clay-Sand pockets
Very fine sand.
Coarse sand & fine gravel.
Clay with soil
Brownish Clay
Sandy Clay
3768.5
Material replaced as originally found.
18'
6'6"
Elev.3769.1
No.2.
Salt grass sod.
Clay and black soil
Brown Clay
Brown Clay with Sand.
Black loam soil
Sand
Coarse sand & fine gravel.
3768.5
Material replaced as originally found.
18'
9'3"
Elev.3769.0
No.3.
Salt grass sod.
Black loam soil
Sand and Clay shoveled in without sorting
Coarse sand & fine gravel.
3768.6
Elev.3770.6
No.4.
Salt grass sod.
Mixture of Clay, Soil, and fine Sand.
Coarse sand & fine gravel.
3769.2
Elev.3770.9
No.5.
Salt grass sod.
Mixture of Clay, Soil, and fine Sand.
Coarse sand & fine gravel.
3769.4
Elev.3771.0
CROSS-SECTIONS OF TANK SETS.
SCALE OF FEET
0 5 10 20
No.6
Salt grass sod.
Mixture of Clay, Soil, and fine Sand.
Coarse sand & fine gravel.
3769.1
Elev.3770.9
No.7
Salt grass sod.
Mixture of Clay, Soil, and fine Sand.
Coarse sand & fine gravel.
3769.2
Elev.3770.8
Inside diam. 7'5¼"
Area 43.44 sq. ft. Soil Tank.
PLAN
Waste nuts riveted and soldered.
Inside diam. 2'4⅜"
Area 4.33 sq. ft. Reservoir Tank.
No.18 Gauge Galv. Iron
SECTION.—
6'-6"
5"
All joints are water-tight.
1/16" Diam. holes
1½" 1½"
Detail of Branch Pipes showing Perforations.
¼" dia. Rod.
Detail of Rim.
Tank Sets 1,2,3, & 5 have ¾" I.D. pipe.
" " 4,6, & 7 " 1" " "
PLAN AND SECTION OF TANKS, SHOWING STANDARD DETAILS.
SCALE OF FEET
0 2 5 10 20
Fence
4' Diam. Well. 3" Casing
Water Evap. Tank.
Rain Gauge No.17.
Water Evap. Pan.
3776
3775
3774
Gate
MAP SHOWING
LOCATION OF SOIL EVAPORATION TANK SETS.
Note:- The area enclosed by fence is in Section 15, T 13 S, R 35 E, M.D.M., near the north quarter corner. Elevations are referred to L.A.A. datum.
MECHANICAL DETAILS OF TANKS
AND
CROSS-SECTIONS AND LOCATION OF TANK SETS
USED IN
SOIL EVAPORATION EXPERIMENTS
NEAR INDEPENDENCE
Notes:- The purpose of the experiments is to measure the evaporation from ground surfaces in the Independence Basin under varying conditions of soil, depth to ground-water, vegetation, etc. The method is to feed the soil in the large tanks from the bottom by supplying water to the reservoir tanks in measured quantities. As soon as capillary action is established, the water is drawn to the surface and evaporates.
Sand in the bottom layer of each soil tank was obtained from Pinyon Wash, and all sizes less than 1/16" were screened out.

FIG. 8.

them with 2-in. galvanized sheet-iron pipe generously perforated with $\frac{1}{16}$-in. holes. These pipes were driven so that the top was just flush with the ground surface, and they were closed at the top with wooden plugs. In some of the tanks it was found impossible to bring the ground-water surface to the desired level with the available hydrostatic pressure from the reservoir tanks, and 2-in. holes were bored between the observation holes to the saturated gravel layer. Water usually rose in these holes to the same height as in the reservoir tank, and, by seeping laterally into the soil, built up the ground-water surface. It was found difficult to keep these holes open to the gravel, however, and the water level in most of them eventually represented the ground-water surface.

Three tank sets were installed in the open valley floor east of Independence in February, 1909. The surface of Soil Tank No. 1 was bare sand; Nos. 2 and 3 (Fig. 7) were laid with salt-grass sod. The initial plan formulated for Tank Sets Nos. 1 and 3 was to hold the ground-water level at various depths below the ground surface for periods of a few weeks during the summer while the climatic conditions were constant, in order to obtain, in a short time and with few tanks, trustworthy results of a general nature. The movement of the water surface from one level to another consumed so much time, however, that winter approached before the experiments on the lower levels were reached, and furthermore, there was no accurate method of determining the volume of evaporated water represented by the differences in depth. The experience of the first year's work with these tanks showed the necessity of maintaining a fixed ground-water level during a complete cycle of climatic changes. In Soil Tank No. 2 it was at first proposed to hold the ground-water level at or near the ground surface, but so great was the rate of summer evaporation that this plan was found to be impracticable with the equipment available. To remedy the defect, the hydrostatic pressure from the reservoir tank was increased by soldering to it a 3-ft. extension, but this was not used until late in the season. This experience suggested the desirability of placing the reservoir tanks above the soil tanks and of increasing the size of the feed pipe.

As a result of these preliminary observations, four additional tank sets were installed in January, 1910. The reservoir tank outlets were placed about 1.7 ft. above the soil tank inlets, and 1-in. pipe was used

throughout. The new soil tanks were laid with salt-grass sod, which took root and grew in every tank. The general plan of operation for Tank Sets Nos. 2 to 7 was to supply the reservoir tanks with water in quantities such that the depths to ground-water in the soil tanks were, respectively, 5 ft., 4.5 ft., 4 ft., 3 ft., 2 ft., and 1 ft. Observations were carried on continuously on the six tanks during the two years, 1910

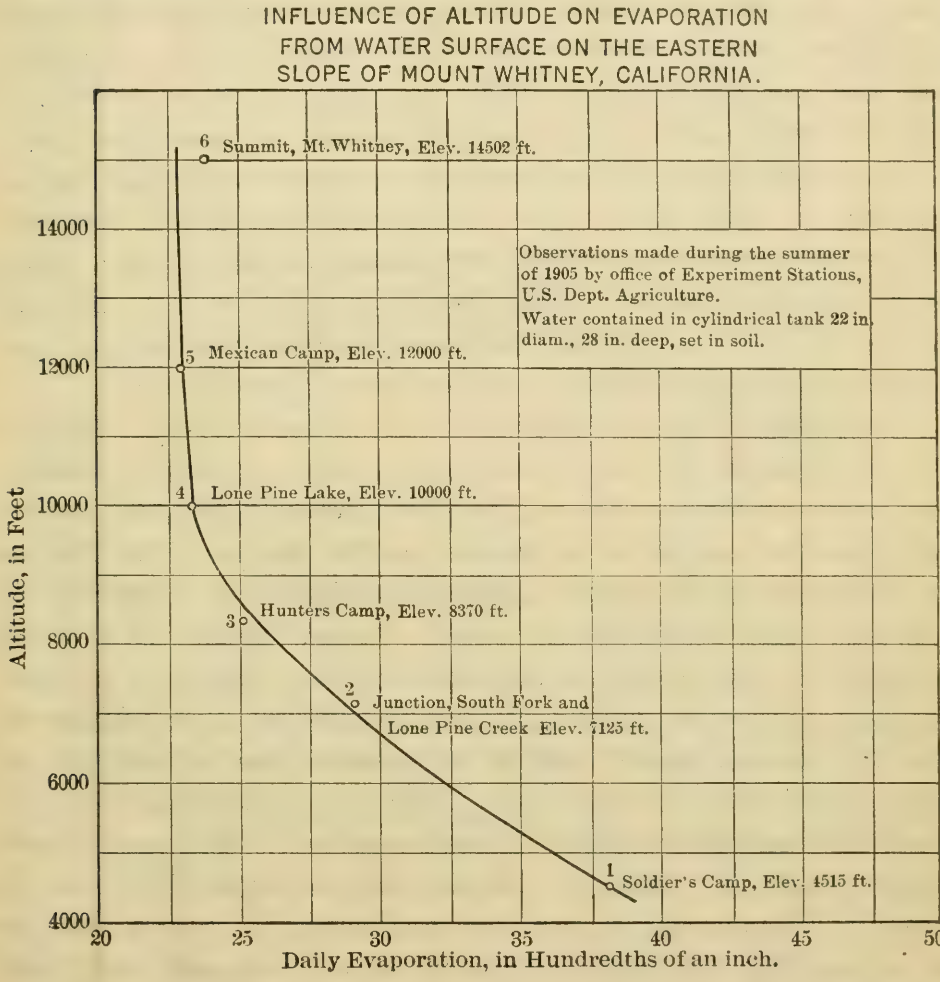


Fig. 9.

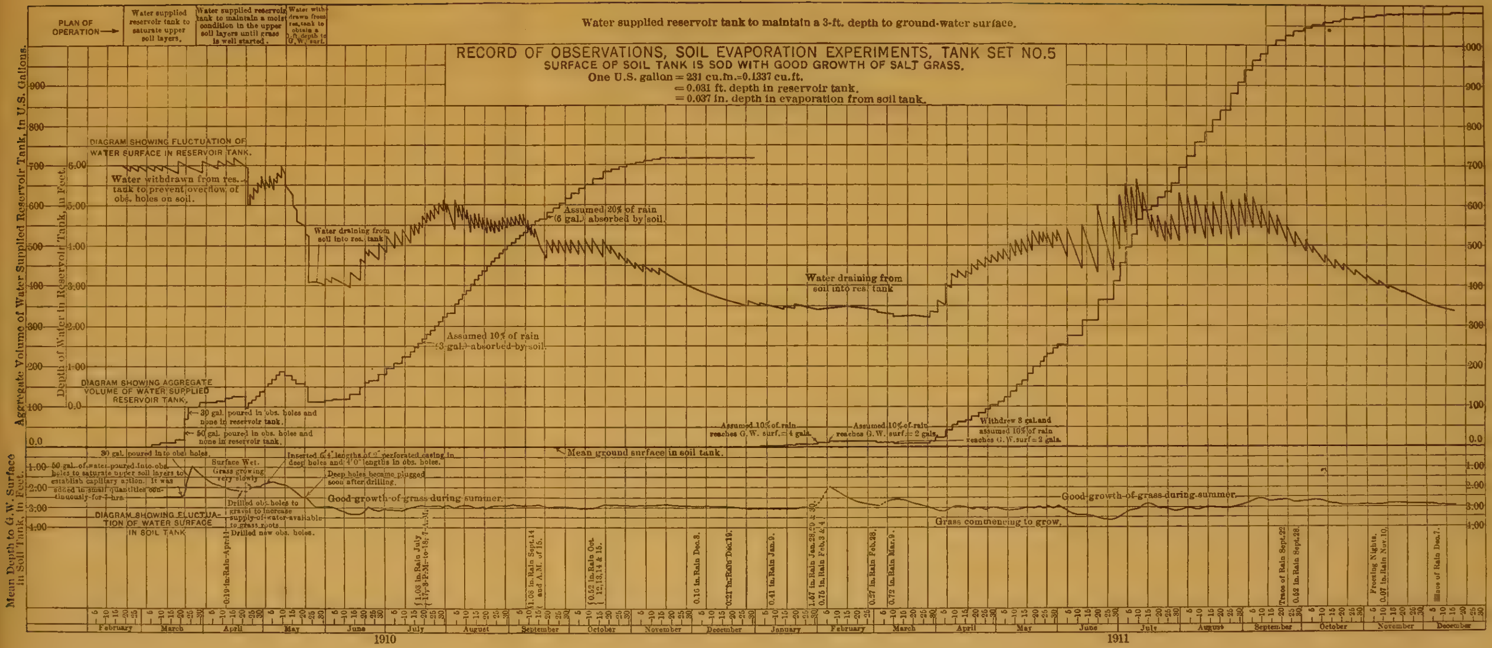
and 1911. The operation and results were quite satisfactory, with the exception that in Tank Set No. 7 the pressure from the reservoir tank was not sufficient between April and September to hold the water level at the 1-ft. depth.

An important feature of reproducing natural conditions for combined soil evaporation and transpiration is to obtain a fully developed root system reaching down to the ground-water surface. At best, this

requires more than a year, particularly for the greater depths. In order to stimulate the growth as much as possible, the water level in all soil tanks was brought up to about 1 ft. below the surface as soon after installation as possible. This was accomplished by pouring water into the observation holes until the soil was completely saturated to the level desired and the surface showed moisture. Then no water was added to the reservoir tanks until the ground-water level had receded by evaporation and transpiration to the desired level. The grass roots were thus given a good initial irrigation and an opportunity to follow the water down. Active growth occurred in Tanks Nos. 4 to 7 during the first year, and continued with greater vigor during the second year. In Tank No. 3 a less active growth occurred the first year, but the results were more satisfactory during the second year. There was practically no growth in Tank No. 2 during the first year, although the grass did not die. During the second year the grass showed more signs of life, but did not grow as actively as in Tank No. 3.

The details of the observations on soil evaporation for 1910 and 1911 for Tank Set No. 5 are shown graphically in Plate V. The supply of water available to the soil from the reservoir tank is the element under complete control of the observer, and at the top of the diagram are statements of the purpose governing additions to or withdrawals from this supply during various periods of time. Below this is platted a broken line representing the fluctuation of water surface in the reservoir tank, the vertical portions indicating additions to or withdrawals from the reservoir supply made by the observer, and the inclined portions indicating the soil-tank draft. There is also platted a mass-curve showing the aggregate volume of water supplied to the reservoir tank, which appears as a series of vertical and horizontal lines. At the bottom of the diagram is platted an undulating line representing the fluctuation of ground-water surface in the soil tank, each depth being obtained by averaging the depths recorded in the observation holes.

The small part that precipitation plays in ground-water fluctuations in Owens Valley is shown by this diagram. The average annual precipitation at the experiment station is about 4.38 in., the season, 1909-10, being normal and 1910-11 well above normal. At the bottom of the diagram is noted the date and quantity of precipitation for each storm. It is seen that, even in a wet season, percolating water does not penetrate to depths exceeding 2.5 ft., unless more than 1 in. falls


PLAN OF OPERATION
Water supplied reservoir tank to saturate upper soil layers.
Water supplied reservoir tank to maintain a moist condition in the upper soil layers until grass is well started.
Water supplied reservoir tank to maintain a 3-ft. depth to ground-water surface.
RECORD OF OBSERVATIONS, SOIL EVAPORATION EXPERIMENTS, TANK SET NO.5
SURFACE OF SOIL TANK IS SOD WITH GOOD GROWTH OF SALT GRASS.
One U.S. gallon = 231 cu.in. = 0.1337 cu.ft.
= 0.031 ft. depth in reservoir tank.
= 0.037 in. depth in evaporation from soil tank.
Aggregate Volume of Water Supplied Reservoir Tank, in U.S. Gallons.
Depth of Water in Reservoir Tank, in Feet.
DIAGRAM SHOWING FLUCTUATION OF WATER SURFACE IN RESERVOIR TANK.
Water withdrawn from res. tank to prevent overflow of obs. holes on soil.
Water draining from soil into res. tank
Assumed 20% of rain (6 gal.) absorbed by soil.
Assumed 10% of rain (3 gal.) absorbed by soil.
Water draining from soil into res. tank
DIAGRAM SHOWING AGGREGATE VOLUME OF WATER SUPPLIED RESERVOIR TANK.
30 gal. poured in obs. holes and none in reservoir tank.
50 gal. poured in obs. holes and none in reservoir tank.
30 gal. poured into obs. holes.
Assumed 10% of rain reaches G. W. surf. = 4 gals.
Assumed 10% of rain reaches G. W. surf. = 2 gals.
Withdrew 8 gal. and assumed 10% of rain reaches G. W. surf. = 2 gals.
Mean ground surface in soil tank.
Mean Depth to G. W. Surface in Soil Tank, in Feet.
50 gal. of water poured into obs. holes to saturate upper soil layers to establish capillary action. It was added in small quantities continuously for 7 hrs.
Surface Wet. Grass growing very slowly
Inserted 6'4" lengths of 2" perforated casing in deep holes and 4'0" lengths in obs. holes.
Deep holes became plugged soon after drilling.
Drilled obs. holes to gravel to increase supply of water available to grass roots.
Drilled new obs. holes.
DIAGRAM SHOWING FLUCTUATION OF WATER SURFACE IN SOIL TANK
Good growth of grass during summer.
Good growth of grass during summer.
Grass commencing to grow.
0.19 in. Rain April 1
1.08 in. Rain July 17, 8 P.M. to 18, 7 A.M.
1.06 in. Rain Sept. 14 and A.M. of 15.
0.52 in. Rain Oct. 12, 13, 14 & 15.
0.16 in. Rain Dec. 8.
0.21 in. Rain Dec. 19.
0.41 in. Rain Jan. 9.
1.67 in. Rain Jan. 28, 29 & 30.
0.75 in. Rain Feb. 3 & 4
0.37 in. Rain Feb. 28.
0.72 in. Rain Mar. 9.
Trace of Rain Sept. 22
0.52 in. Rain Sept. 28.
Freezing Nights.
0.07 in. Rain Nov. 10.
Trace of Rain Dec. 7.
February
March
April
May
June
1910
July
August
September
October
November
December
January
February
March
April
May
June
1911
July
August
September
October
November
December

within a short period on moist soil. Even then it does not appear to reach depths greater than 4 ft. The problem of percolation from rainfall, therefore, is practically eliminated from the experiments. When rising ground-water was noted in a soil tank after precipitation, the volume of percolating water was estimated from the observed rise and included in the mass-curve, as noted on the diagrams.

The quantity of water evaporated from the soil surface of any tank during a given period can be computed accurately from the diagrams, when the depth to ground-water at the beginning and end of the period is the same, by noting from the mass-curve the quantity supplied to the reservoir tank during the period and the accumulation or depletion in the reservoir tank. The sum of these quantities, with their proper algebraic sign, gives the loss by evaporation. For differing depths to ground-water, however, the computations are only approximate, because the proportion of empty space in the soil layer and the quantity of moisture it contained initially are both unknown. A monthly summary of results for Tank Sets Nos. 2 to 7 for 1911 is presented in Tables 9 to 14. The annual depth of evaporation from the several soil tanks exhibited a consistent decrease with increase of depth to ground-water, and varied from 48.8 in. for No. 7 to 13.43

TABLE 9.—Depth of Evaporation From Ground Surface Near Independence During 1911.

Tank Set No. 2.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan.	6	1.10	1.15	+ 2	4	0.15	0.005	4.98
Feb.	0	1.15	1.11	— 1	1	0.04	0.001	4.95
Mar.	3	1.11	1.14	+ 1	2	0.07	0.002	4.94
Apr.	10	1.14	1.18	+ 1	9	0.33	0.011	4.94
May	30	1.18	1.19	0	30	1.11	0.036	4.98
June	66	1.19	1.39	+ 6	60	2.22	0.074	5.03
July	71	1.39	1.22	— 5	76	2.81	0.091	5.00
Aug.	105	1.22	1.78	+ 18	87	3.22	0.104	4.95
Sept.	52	1.78	1.52	— 8	60	2.22	0.074	4.80
Oct.	18	1.52	1.37	— 5	23	0.85	0.028	4.80
Nov.	1	1.37	1.18	— 6	7	0.26	0.009	4.85
Dec.	1	1.18	(1.10)	— 3	4	0.15	0.005	4.98
Year	363	1.10	1.10	0	363	13.43	0.037	4.94

TABLE 10.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 3.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan........	9	0.14	0.10	— 1	10	0.37	0.012	4.53
Feb........	18	1.10	0.23	+ 4	14	0.52	0.019	4.59
Mar........	12	0.23	0.20	— 1	13	0.48	0.015	4.48
Apr........	40	0.20	0.54	+ 11	29	1.07	0.036	4.50
May	62	0.54	0.60	+ 2	60	2.22	0.072	4.49
June.......	103	0.60	0.96	+ 12	91	3.37	0.112	4.63
July.......	160	0.96	0.67	— 9	169	6.25	0.202	4.51
Aug........	285	0.67	2.00	+ 43	242	8.96	0.289	4.52
Sept........	135	2.00	1.13	— 28	163	6.03	0.201	4.22
Oct........	27	1.13	0.54	— 19	46	1.70	0.055	4.21
Nov........	4	0.54	0.16	— 12	16	0.59	0.020	4.33
Dec	3	0.16	(0.10)	— 2	5	0.18	0.006	4.56
Year.....	858	0.14	0.10	0	858	31.74	0.087	4.46

TABLE 11.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 4.

Month.	Volume of water supplied to reservoir tank in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate in inches per 24 hours.	
Jan........	7	1.53	1.49	— 1	8	0.30	0.010	3.97
Feb........	— 8	1.49	1.18	— 10	2	0.07	0.002	3.36
Mar........	3	1.18	1.19	0	3	0.11	0.004	3.74
Apr........	62	1.19	2.15	+ 31	31	1.15	0.038	4.00
May........	96	2.15	2.57	+ 14	82	3.04	0.098	4.06
June.......	121	2.57	2.63	+ 2	119	4.40	0.147	(3.34)
July.......	114	2.63	1.97	— 21	135	5.00	0.161	3.44
Aug........	141	1.97	2.47	+ 16	125	4.63	0.149	3.92
Sept.......	65	2.47	1.90	— 18	83	3.07	0.102	3.85
Oct........	30	1.90	1.53	— 12	42	1.55	0.050	3.93
Nov........	9	1.53	1.09	— 14	23	0.85	0.028	3.97
Dec........	6	1.09	(0.91)	— 6	12	0.44	0.015	4.10
Year.....	646	1.53	0.91	— 19	665	24.61	0.067	3.81

TABLE 12.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 5.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan	8	2.55	2.43	— 4	12	0.44	0.014	3.01
Feb	6	2.43	2.42	0	6	0.22	0.008	2.47
Mar	3	2.42	2.36	— 2	5	0.18	0.006	2.73
Apr	88	2.36	3.65	+ 42	46	1.70	0.057	2.99
May	141	3.65	4.09	+ 14	127	4.70	0.151	3.01
June	166	4.09	4.19	+ 3	163	6.03	0.201	3.40
July	244	4.19	4.14	— 2	246	9.11	0.294	3.06
Aug	255	4.14	4.92	+ 23	232	8.58	0.280	2.99
Sept	127	4.92	4.07	— 25	152	5.62	0.188	2.69
Oct	36	4.07	3.24	— 27	63	2.33	0.075	2.77
Nov	9	3.24	3.67	— 18	27	1.00	0.033	2.83
Dec	1	2.67	(2.45)	— 7	8	0.30	0.010	2.90
Year	1 084	2.55	2.45	— 3	1 087	40.21	0.110	2.90

TABLE 13.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

TANK SET NO. 6.

Month.	Volume of water supplied to reservoir tank, in gallons.	Depth of Water in Reservoir Tank, in Feet.		Accumulation or depletion of water in reservoir tank, in gallons.	Volume of Water Evaporated.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan	12	3.03	3.19	+ 5	7	0.26	0.008	1.92
Feb	9	3.19	3.17	— 1	10	0.37	0.013	1.31
Mar	24	3.17	3.32	+ 5	19	0.70	0.023	1.70
Apr	94	3.32	3.53	+ 7	87	3.22	0.107	2.00
May	163	3.53	3.54	0	163	6.03	0.195	1.99
June	182	3.54	3.75	+ 7	175	6.48	0.216	2.30
July	213	3.75	3.31	—14	227	8.40	0.271	2.02
Aug	314	3.31	4.25	+30	284	10.50	0.339	2.01
Sept	143	4.25	3.89	—12	155	5.74	0.192	1.55
Oct	38	3.89	3.36	—17	55	2.04	0.065	1.65
Nov	8	3.36	2.98	—12	20	0.74	0.025	1.84
Dec	6	2.98	(2.83)	— 5	11	0.41	0.013	2.04
Year	1 206	3.03	2.83	— 7	1 213	44.89	0.122	1.86

TABLE 14.—DEPTH OF EVAPORATION FROM GROUND SURFACE NEAR INDEPENDENCE DURING 1911.

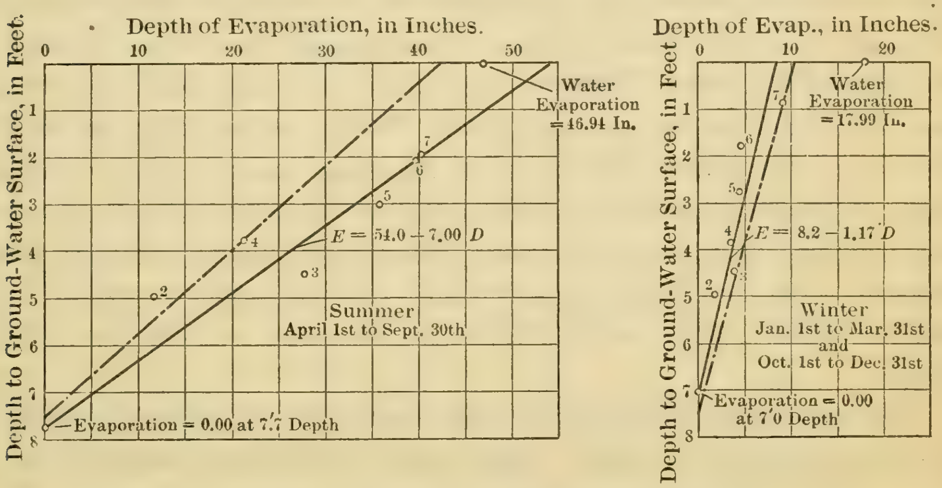
TANK SET NO. 7.

Month.	Volume of water supplied to reservoir tank, in gallons.	DEPTH OF WATER IN RESERVOIR TANK, IN FEET.		Accumulation or depletion of water in reservoir tank, in gallons.	VOLUME OF WATER EVAPORATED.			Average depth to ground-water surface in soil tank, in feet.
		Beginning of month.	End of month.		Total, in gallons.	Depth, in inches.	Rate, in inches per 24 hours.	
Jan........	3	5.07	4.70	— 12	15	0.56	0.018	0.73
Feb........	31	4.70	5.08	+ 12	19	0.70	0.025	0.81
Mar........	67	5.08	5.90	+ 26	41	1.52	0.049	0.98
Apr........	102	5.90	5.96	+ 2	100	3.70	0.123	1.46
May	151	5.96	5.92	— 1	152	5.62	0.181	1.64
June.......	160	5.92	5.62	— 10	170	6.30	0.210	2.06
July.......	223	5.62	5.40	— 7	230	8.52	0.275	2.19
Aug........	255	5.40	5.99	+ 19	236	8.74	0.281	2.51
Sept.......	178	5.99	5.79	— 6	184	6.81	0.227	2.39
Oct........	115	5.79	5.77	— 1	116	4.29	0.139	1.44
Nov........	14	5.77	4.96	— 26	40	1.48	0.049	0.60
Dec........	0	4.96	(4.50)	— 15	15	0.56	0.018	0.60
Year.....	1 299	5.07	4.50	— 18	1 318	48.80	0.133	1.45

in. for No. 2. The depth of summer evaporation varied from 81 to 90% of the annual in the several tanks, and averaged 87 per cent. The month of maximum evaporation is August, and minimum evaporation occurs during December to March, inclusive. The exact date of maximum evaporation rates for the several soil tanks occurs about September 1st, and they follow each other consecutively with greater depth to ground-water. The approximate dates of maximum and minimum air temperatures at Independence are July 10th and January 10th, respectively, but no measurements were made to determine the lag of corresponding soil temperatures at various depths. The extremes of evaporation from water surfaces agree in time of occurrence with maximum and minimum air temperatures, however, and the observed lag in soil evaporation is in general consistent with the observed lag in soil temperatures in other localities. Hence it is reasonable to conclude that extremes in the rate of soil evaporation and soil temperature are concurrent at a given depth.

A graphic study of the data in Tables 9 and 14 for the periods, April 1st to September 30th, and October 1st to March 31st, which are, respectively, periods of increasing and decreasing evaporation rate, is presented in Fig. 10. There appears to be, during each period,

a straight-line relation between total evaporation and depth to ground-water. The limiting depth is 7.7 ft., and the total evaporation when water and ground surface coincide, 54.0 and 8.2 in., respectively. The total depth of evaporation, in inches, being represented by E, and the depth to ground-water, in feet, by D, the equations representing variation in evaporation with depth to ground-water are $E = 54.0 - 7.00\ D$ and $E = 8.2 - 1.17\ D$. It will be noted that the combined soil evaporation and transpiration during the summer exceeded the water evaporation from the tanks in the soil by 15 per cent.



OWENS VALLEY SOIL EVAPORATION CURVES
FOR ALKALI SALT GRASS LAND
Based on Soil Tank Observations of 1911.
——— Curve based on 1911 Data.
-——- Curve based on 1910-11 Data.
o 1911 Observations.

FIG. 10.

The curve based on the 1910-11 data is shown on the diagram for comparison. There was a marked increase in total evaporation during 1911, as compared with the season, 1910-11, which was due to the more complete development of the root systems. Broadly speaking, the results for 1911 showed an increase of 17% in the volume of water consumed during the two periods. A continuation of observations for another year would show still further increase for those tanks in which the grass roots had not reached the water plane. Observations of depth to water in test holes in the transition zone between meadow and desert land indicate that soil evaporation ceases for depths exceeding 8 ft. The effect of increased evaporation for the tanks with greatest depth to ground-water would be to drop the lower end of the

curve to some point below 7.7 ft. The true curve, therefore, is probably steeper than that for 1911, crossing the X-axis at about the same point, but the Y-axis at about 8 ft. instead of 7.7 ft. However, in the practical use of the curve, the departure of the lower end from the true position does not affect materially the computations of the total volume of water evaporating from a given area, as the proportion of such volume originating in areas of relatively deep ground-water is small.

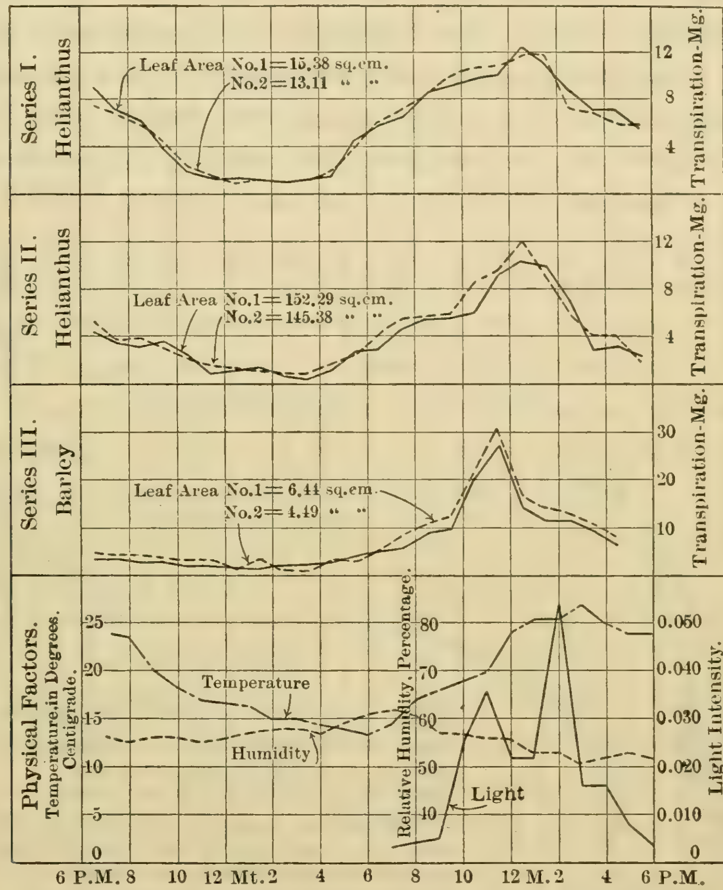
Transpiration.—A considerable portion of the water evaporating from the soil is absorbed by plant roots and carried upward through the stem and into the foliage, whence it escapes in the process of transpiration. This process continues as long as the plant has life, but is most active during the growing period. Transpiration differs in different species of plants, and even in the same species when existing under different conditions of light, atmospheric pressure, soil texture, and available moisture in the soil. King's experiments indicate that humidity does not affect transpiration.* For a species growing in a definite locality, light and available soil moisture are the controlling factors.

The process of transpiration and respiration in plants is similar to the breathing of animals. Both plants and animals inhale air and exhale from the respiratory organs large quantities of water. The lungs of animals are intended primarily to provide a means for the entrance of oxygen into the body and for the escape of carbon dioxide, but they cannot perform their functions unless the interior lining of the air cells is kept moist. Similarly, the breathing surface of a plant must be kept moist, and, as a protection from too rapid evaporation, this surface is within the plant structure, principally in the foliage. Plant leaves are enclosed in a relatively impervious skin or epidermis in which are small breathing pores or stomata which open or close automatically, depending on the needs of the plant for a greater or less quantity of air. When exposed to light, the food-manufacturing processes of a green plant are stimulated, and require a continually changing volume of air in contact with the breathing surface. The stomata open proportionally to the light intensity. Should the water supply in contact with the roots be insufficient, the breathing surface may become dry, and when that happens the stomata close automatically

* "Irrigation and Drainage," by F. H. King, New York, 1899.

until the proper quantity of air is admitted for the plant to do its work under the new conditions. The stomata, therefore, control the quantity and rate of loss of water from plants by transpiration.

There is a marked diurnal periodicity in the rate of transpiration, which investigators are led to believe is largely the result of the varying intensity of light. This periodicity is well illustrated by ob-



RELATION OF TRANSPIRATION TO LIGHT INTENSITY DURING A 24-HOUR DAY.

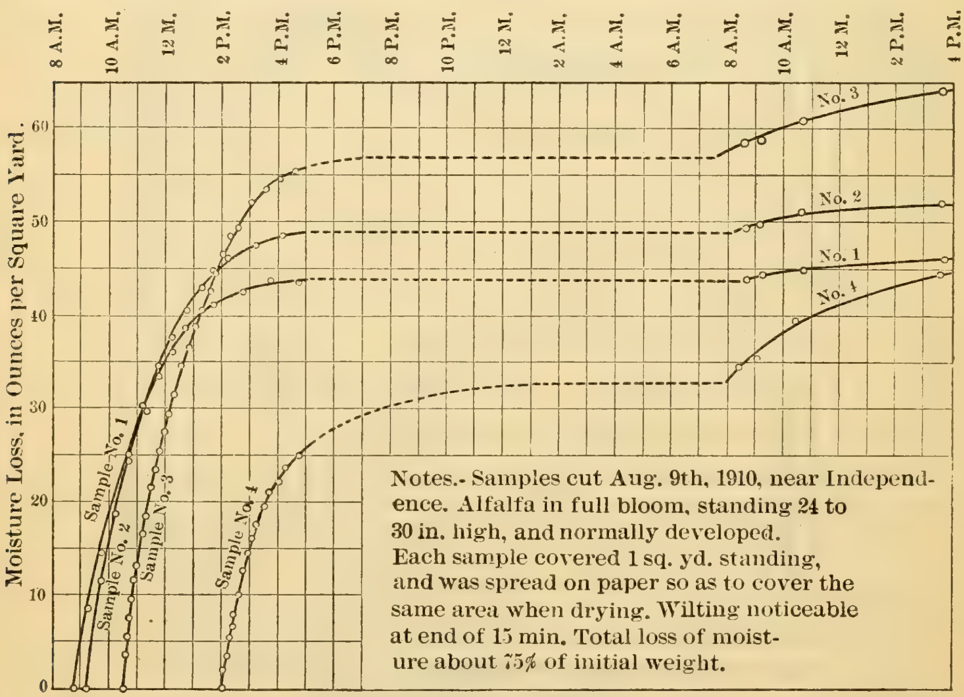
FIG. 11.

servations made under the direction of Mr. Frederick E. Clements, State Botanist of Minnesota, and reproduced in Fig. 11.* Measurements of transpiration were made hourly from 6 P. M. on February 16th to 6 P. M. on February 17th, and the physical factors were observed between these hours. The day was cloudy throughout, so that

* "Influence of Physical Factors on Transpiration", by A. W. Sampson and L. M. Allen, Minnesota Bot. Studies, Pt. I, Vol. 4, 1909, p. 42.

the variation in temperature and humidity was slight. The diagrams show very strikingly the response of transpiration to changes in intensity of light.

No measurements of transpiration are available for conditions similar as regards altitude and aridity to those in Owens Valley. It is unnecessary in this study to know separately the transpiration from wild grasses and the evaporation from bare soil, because the area of the latter is relatively small. The experiments on soil evaporation, therefore, were planned to give the combined loss from these two causes. It is desirable, however, to know the quantity of transpiration from field crops, in order to aid in computing the quantity of percolation from irrigation. Observations for such crops were confined to alfalfa.



RATE OF LOSS OF WATER FROM FRESHLY CUT ALFALFA, BASED ON FIELD AREAS.

FIG. 12.

The method of measurement was based on the assumption that the rate of loss of water from freshly cut plants would correspond closely to the rate before cutting. The plants were cut rapidly from a measured area, weighed, and spread out on paper to cover the same area as before cutting. At short intervals they were reweighed until there was no further appreciable loss. No noticeable wilting occurred during the first 15 min. and the rate of loss during this period was used as

a basis for calculations. The results of the measurements are shown graphically in Fig. 12.

The rapid decrease in the rate of loss is very noticeable. Inspection of Fig. 11 will show that the rates of transpiration at 8.45, 9.15, and 10.30 A. M., and 2.02 P. M., expressed as percentages of the average rate for 24 hours, are, respectively, 128, 141, 177, and 197, an average of 161 per cent. If a similar relation is assumed, the average loss in a 24-hour day from the four alfalfa samples would be 366 oz. per sq. yd. of field area, or 0.49 in. in depth. This figure appears to be rather large, at first glance, for the rate of evaporation for that day from the pan in Owens River was 0.30 in., and that from the shallow pan in the soil was 0.38 in. The results obtained by German investigators indicate the loss from sod during the growing season to be 92% greater than from water surface, and that from cereals to be 73% greater. Furthermore, the humidity of the air after passing over an alfalfa field is very noticeably greater than after crossing a body of water. The result obtained in the experiment here described, therefore, is within reason.

The growing season for alfalfa in the vicinity of Independence is marked by an entire absence of cloudiness. It extends from about April 15th to September 30th, during which time three crops mature, the yield being about 5 tons of dry matter to the acre. The samples used for the experiments were almost ready for the second cutting. On the assumption that the average area of transpiring surface during the entire growing season was 50% of that on the day of the experiment, the total loss of water during the season would amount to 41 in., or 3.43 ft. in depth. Therefore, with a production of dry hay, amounting to 5 tons to the acre, there would be 1 lb. of dry matter for every 93.5 lb. of water lost by transpiration.

Ground-Water.

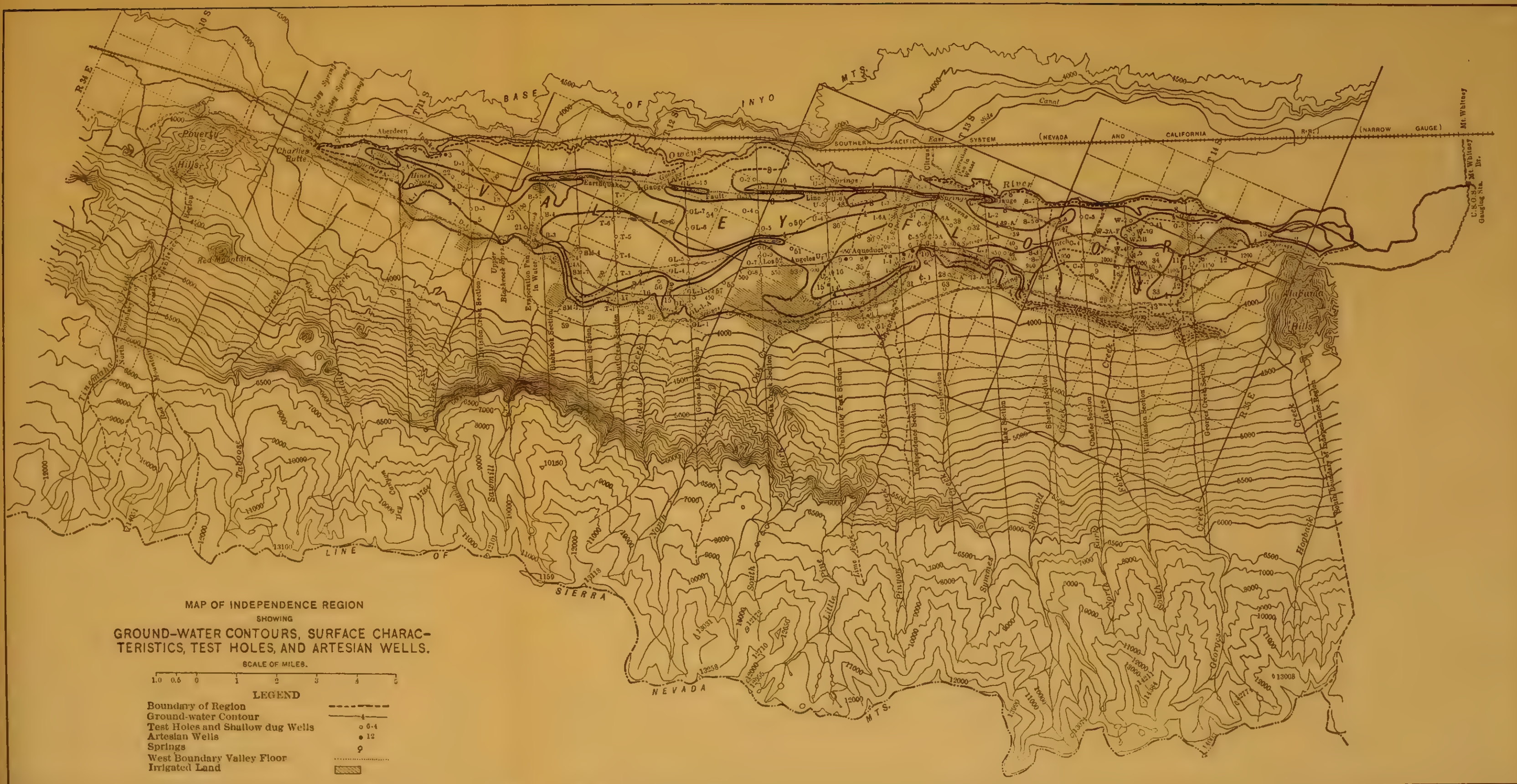
Form of the Ground-Water Surface.—The general form of the ground-water surface corresponds with that of the surface of the valley fill, although the slopes are less steep and the irregularities are not so pronounced. In the valley floor the depth to ground-water is only a few feet. It becomes progressively greater toward the mountains, and probably lies 200 or 300 ft. beneath the outwash slope at about the 5 000-ft. contour. Superimposed on the general ground-water surface are sharp "ridges" beneath stream channels and "mounds" under irrigated

fields. The surface of the water in the underground reservoir, therefore, is not a level plain, but has a varied topography.

There are two reasons for this condition: the action of gravity tending to equalize inequalities in the ground-water surface, and the resistance which the ground offers to the lateral motion of water through its interstices. Percolating waters enter the valley fill from the upper edge of the outwash slope, from stream channels crossing the outwash slope, and from irrigated fields. The valley floor is the lowest portion of the valley fill and also the ground-water outlet. The force of gravity, therefore, tends to draw percolating water which has reached the surface saturation to the level of the valley floor. This can occur only by a lateral movement of water from the outwash slope toward the valley floor, but the resistance of the porous material is so great that a steep gradient is necessary to maintain even a very low velocity. Hence there is the steep slope of the ground-water surface from the mountains toward the valley, at many points exceeding 80 ft. per mile, and laterally from stream channels and irrigated fields. The lateral movement of the water is so slow that percolating water entering at the upper edge of the outwash slopes, does not reach the valley for at least 2 years.

In order to outline the ground-water definitely, all existing domestic wells in the region were located and many additional observation wells were drilled, where the cost was not prohibitive. The region contains 27 domestic wells, 12 of which are on the valley floor and 15 on the outwash slopes. There were drilled in addition 142 observation wells, all but two being on the valley floor. These wells were sufficient to define the ground-water surface over about 60 sq. miles of the region.

Ground-water contours for the valley floor showing lines of equal average depth to ground-water have been worked out on Plate VI. They represent the average position of the surface of saturation between the annual extremes. The data are sufficient to determine the 3-, 4-, and 8-ft. contours with reasonable accuracy. The sudden approach of ground-water toward the surface at the upper edge of the grass land is shown, and also the general proximity of ground-water to the surface throughout the valley floor. The total area between the westerly 8-ft. contour and Owens River is 67 sq. miles. The average depth to ground-water is between 4 and 8 ft. over 40% of this area, and between 3 and 4 ft. over 28 per cent. It exceeds 8 ft. over 14% of the area, and is 3 ft. or less over 18 per cent. The area of the valley floor is 66.4 sq. miles


MAP OF INDEPENDENCE REGION
SHOWING
GROUND-WATER CONTOURS, SURFACE CHARACTERISTICS, TEST HOLES, AND ARTESIAN WELLS.
SCALE OF MILES.
LEGEND
Boundary of Region
Ground-water Contour
Test Holes and Shallow dug Wells
Artesian Wells
Springs
West Boundary Valley Floor
Irrigated Land
BASE OF INYO MTS.
Owens River
VALLEY FLOOR
SOUTHERN PACIFIC East SYSTEM (NEVADA AND CALIFORNIA R.R.) (NARROW GAUGE)
LINE OF SIERRA NEVADA MTS.
Poverty Hills
Red Mountain
Charlies Butte
Aberdeen
Aqueduct
Alabama Hills
Mt. Whitney

(Table 1), and its west boundary practically coincides with the 8-ft. contour.

There is a very striking relation between vegetation and depth to ground-water. On the outwash slopes the vegetation consists of various stunted desert shrubs. In approaching the valley floor at about the 20-ft. contour, sagebrush begins to predominate, and has a luxuriant growth as far east as the 12-ft. contour, where it is replaced by greasewood, rabbit brush, and coarse bunch grass. In the vicinity of the 8-ft. contour, salt grass begins to appear, and farther east, near and within the area inclosed by the 4-ft. contour, it grows luxuriantly. Within the 3-ft. contour, fresh-water grasses thrive where there is sufficient surface water to leach out and carry away most of the alkali, but the salt grass grows well, even where the soil is alkaline. In various portions of the valley floor rabbit brush and greasewood are found where the average depth to ground-water is 4 ft. or more, but grass predominates east of the 8-ft. contour. In areas where the alkali is excessive there is practically no vegetation. In general, grass does not grow where the depth to ground-water exceeds 8 ft., so the 8-ft. contour tends to coincide with the boundaries between meadow and desert lands.

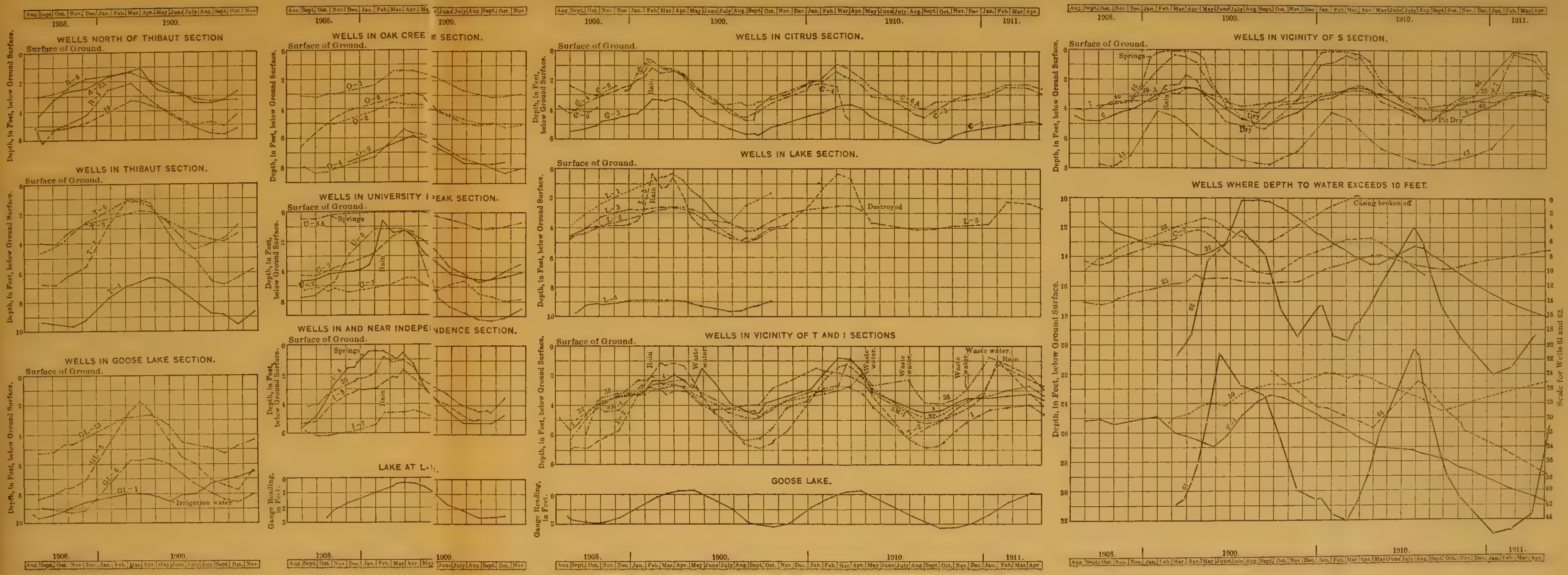
Fluctuation of the Ground-Water Surface.—The surface of the ground-water is continually fluctuating. Both the extent and character of this fluctuation vary widely in different localities and at different times, depending on the proximity to ground-water sources or outlets and the relative rates of ground-water accretion and depletion. Three pronounced types are to be observed in the Independence Basin: (1) broad irregular fluctuations of varying amplitude in the outwash-slope area; (2) slightly irregular periodic fluctuation with wide fixed limits in and near irrigated areas; and (3) a regular periodic fluctuation with comparatively narrow and fixed limits in the valley floor. Special characteristics are also exhibited by wells within certain limited areas, as the result of local ground-water conditions.

These fluctuations were determined and studied from well observations made by the methods already described. Readings obtained at intervals of from 2 to 4 weeks were sufficient to establish accurately the position of the ground-water surface, as the fluctuations are characterized by great regularity. Most of the wells were observed from August 15th, 1908, to November 15th, 1909, and on 26 of the most

typical wells observations were continued to May 1st, 1911. The fluctuation of the surface of the lake south of Citrus Bridge was observed from August 15th, 1908, to November 15th, 1909, and of Goose Lake from August 15th, 1908, to May 1st, 1911.

The type of fluctuation peculiar to wells on the outwash slope is shown in Plate VII by Wells Nos. 31, 64, 25, 26, and 59, and Citrus No. 1. Water stands 10 ft. or more below the surface in all these wells, the vegetation of the surrounding area is limited to desert shrubs, and there are no alkali deposits on the surface. With knowledge of the sources and movements of ground-water beneath the outwash slopes, the assigned cause for this type would be annual variation in the quantity of water supplied by percolation from precipitation on the intermediate and outwash slopes and from stream channels. This is confirmed by the observations. For example, Well No. 31, which is 7 miles from the base of the Sierra and 500 ft. south of the old channel of Pinyon Creek, exhibits a persistent downward tendency which was partly checked during the summer of 1909 and 1910. The maximum effect of the very wet years, 1906 and 1907, evidently reached this well in 1908 and early in 1909. During the following years the water had a tendency to return to its normal level. This was twice opposed by percolation from the channel of Pinyon Creek, which carried flood-water during a few weeks in June and July, 1909, and for a very short period in 1910. Citrus Well No. 1, which is about ¾-mile south of Well No. 31, has similar fluctuations, but in it the maximum effect of seepage from Pinyon Creek is registered 6 weeks later in 1909, and in 1910 is much smaller in quantity. Well No. 64, situated similarly with respect to the mountains, but north of Little Pine Creek, has the same downward tendency, which is checked temporarily during the summer by irrigation in a near-by alfalfa field and a small garden at the well. Well No. 59, which is 2 miles from the base of the Sierra and ½ mile south of Sawmill Creek, had an upward tendency during 1909, due to the percolation from precipitation of the wet winter, 1908-09. In 1910 the water level fell in response to the normal winter of 1909-10. Seepage from Sawmill Creek does not affect this well appreciably. Wells Nos. 25 and 26 exhibit the general tendencies of Well No. 59, but they are in the transition zone between the outwash slope and the valley floor, where there is a periodic back-water effect from the annual rise of ground-water in the grass land.

DIAGRAMS SHOWING FLUCTUATION OF GROUND-WATER SURFACE. TYPICAL OBSERVATION WELLS IN INDEPENDENCE BASIN



Wells Nos. 61 and 62 illustrate the type of fluctuation characteristic of the irrigated areas of the region. They are in irrigated gardens in the Town of Independence. The form of the curve is periodic, with sharp crests and troughs, the former in July, the latter in January or February. The fluctuation in such wells ranges from 10 to 20 ft. in different portions of the basin. Irregularities superimposed on the broad periodic curve are the result of irregularity in the application of irrigation water.

The fluctuation of the ground-water surface in various parts of the valley floor, other than at the eight wells already mentioned, is shown by 48 typical well records on Plate VII.

Permanent bench-marks were established at each well, and test holes from which measurements to the water surface could easily be made with a steel tape. Before observing, a weight is fastened at the end and the tape is chalked. The end of the tape is then submerged and the difference between the readings at the bench and at the water surface is the depth. Corrections are made in the office when the bench is not at the ground surface. Readings are to feet and hundredths.

Most of these wells are where the average depth to ground-water is less than 8 ft. The adjoining ground surface is more or less crusted with alkali, and, where the alkali is not too concentrated, several species of wild grass grow vigorously. The two lakes and Wells Nos. C-3 and 43 are in areas where the average depth to ground-water exceeds 8 ft., and desert conditions prevail.

The fluctuations observed in all the valley-floor wells are remarkably uniform. When platted, the observations give smooth and regular curves with an annual periodicity. The average time of occurrence of crests for wells in grass or alkali areas is March 28th; the troughs occur on September 20th, 6 months later. Heavy winter precipitation, or the proximity of springs, advances the crests into January or February, but, in the desert areas, the crest lags into April or May. The fluctuation between maximum and minimum levels in normally situated wells ranges from 1.5 to 4 ft. Wells which are near or below springs in the vicinity of intermittently occurring surface water have a greater range, which may reach 7 ft. The average fluctuation for 1908-09, as observed in 122 wells distributed generally over the valley floor, is 3.14 ft. This average represents normal conditions.

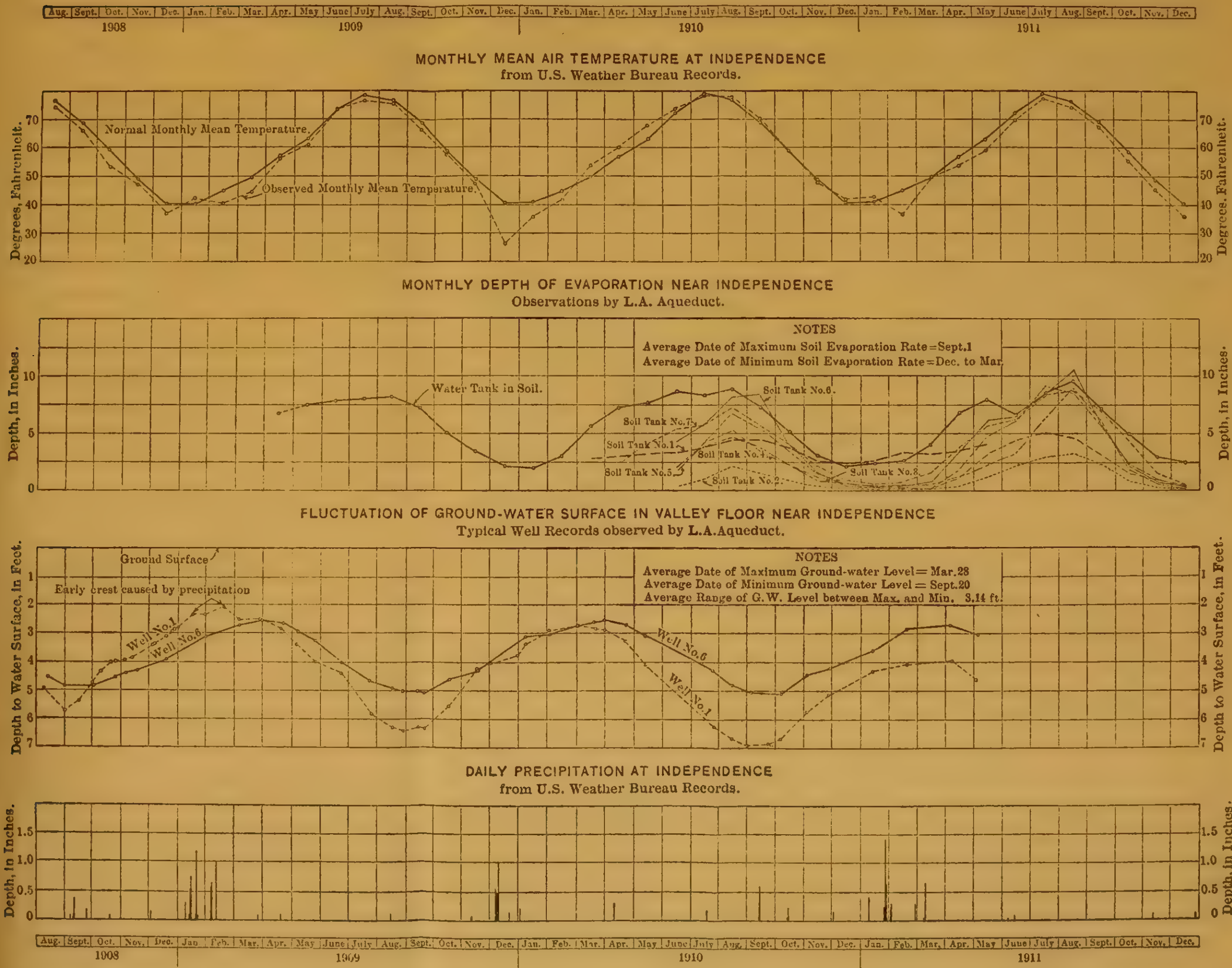
Fluctuation of this type is due to evaporation from the soil and transpiration, processes which are active wherever there is capillary connection between the surface of saturation and the ground surface, or wherever gravity water or capillary water is within reach of plant roots. Two facts have led to this conclusion: (1) the area characterized by capillary connection between ground-water surface and ground surface and by accessibility of ground-water to plant roots is coincident with the area exhibiting this type of periodic fluctuation; and (2) the combined rates of evaporation from soil and transpiration, as observed experimentally, increase and decrease concurrently and in the same ratio with the fall and rise of the ground-water surface.

The first of these facts is indicated by the following observations: Surface incrustations of alkali are now known among investigators to be an indication of evaporation from the soil, and a growth of natural grasses certainly shows the presence of water within reach of plant roots. These manifestations are both strictly confined to valley-floor areas within which the periodic fluctuation is observed. There are valley-floor areas, however, within which the periodic fluctuation occurs, but which have a loose sandy surface devoid of alkali and vegetation. An examination of such areas shows that they are surrounded or bordered by meadow and alkali-crusted land, and further that maximum and minimum ground-water levels exhibit a lag in time of occurrence which varies with the distance from these adjoining lands. (See Plate VII, Well C-3 and Goose Lake.) The fluctuations in these desert areas do not originate within the areas themselves but in the near-by lands, from which they are propagated as annual waves. In general, average depths to ground-water exceed 8 ft. in desert areas but are less than 8 ft. in meadow or alkali lands.

The second fact—that variations of soil evaporation and transpiration are similar to ground-water fluctuations—is indicated by the results of the experiments on evaporation from soil. The maximum rate of soil evaporation occurs about September 1st and the minimum from December 1st to April 1st. The lowest ground-water level occurs about September 20th and the highest level on March 28th (Plate VIII). Thus the critical points in the curves of soil evaporation and ground-water fluctuation are practically coincident as regards time. Furthermore, although the curves are inversely related, their form is remarkably similar. The obvious conclusion is that ground-water fluctuations

PLATE VIII.
PAPERS, AM. SOC. C. E.
APRIL, 1914.
LEE ON
YIELD OF UNDERGROUND RESERVOIRS.

TEMPERATURE, EVAPORATION FROM SOIL, PRECIPITATION AND FLUCTUATION OF GROUND-WATER SURFACE IN VALLEY FLOOR NEAR INDEPENDENCE.



in non-irrigated portions of the valley fill are the result of evaporation from the soil and transpiration.

Variations from the normal periodic curves occur for three causes: large precipitation, seepage from springs, and seepage from standing or flowing surface water. The infrequency of precipitation sufficient to raise the ground-water surface is shown on Plates VII and VIII. It is practically a negligible factor in ground-water fluctuations. The springs at the upper edge of the outwash slope affect ground-water conditions in their vicinity by stimulating the annual rise and maintaining the ground-water level at a maximum during several months prior to March. (See Plate VII, Wells Nos. 46, U-6A, and I-4.) This results from the decrease in the rate of soil evaporation which allows the accumulation of their discharge in the surrounding soil at a greater rate than in adjoining areas where the rate of supply of underground water is less. Surface water has its source in large springs, waste from irrigation, and the flood waters of mountain streams. It occurs at various times and places, and cannot be considered as a permanent factor in ground-water fluctuations. (See Plate VII, Wells Nos. 4, 38, 39-A, 32, and GL-1.) The irregular fluctuations of ground-water on the outwash slope do not appear in the valley floor because of the relief afforded by the escape of water in springs at the upper edge of the grass land.

Ground-Water Losses.—Ground-water fluctuations within the valley floor consist primarily of the regular annual rise and fall produced by variation in the rate of evaporation. This is indicated by actual observations extending over 3 years, and confirmed by the persistency of various perennial plant species. Hence there must be overflow of ground-water from the valley fill of the region equal to the average inflow by percolation. The possible outlets would seem to be underflow southward through the valley fill, underflow by way of deep fissures, seepage and spring flow into the channel of Owens River, evaporation from spring waters, evaporation from damp soil, and transpiration from vegetation. The first two of these are eliminated by the geology and topography of the region. The slope of the ground-water surface in the valley fill opposite the Alabama Hills does not exceed 8 ft. to the mile, and the material is fine sand and clay, as indicated by the Southern Pacific Company's well at Lone Pine Station. Even if there is a movement of ground-water southward from the

region, it must be exceedingly slow, and it would be entirely intercepted by the alluvial fan of Lone Pine Creek, which has a groundwater surface higher than the valley fill to the north. The granitic formation of the Sierra Nevada and the granite core of the Inyo Mountains are complete barriers against the escape of underground waters through any formation but the valley fill. It has already been shown that there is no seepage flow into Owens River from the water supply of the region. Hence the outlets by evaporation, transpiration, and spring discharge into Owens River are all that remain to be considered.

Soil evaporation and transpiration will be considered, first, for irrigated lands, and second, for the general grass and alkali area of the valley floor. The quantity of water used in irrigating the 3 011 acres under cultivation in the region is about 72 sec-ft. of continuous flow for 6 months (Table 18), which is equivalent to a depth of 8.6 ft. over the whole area. The depth of transpiration from alfalfa during the irrigating season has already been computed as 3.43 ft., or 40% of the total volume used. There is also a small loss through evaporation from the soil during and immediately after irrigations, say 0.85 ft., or 10% of the total. The total loss by evaporation from the soil and by transpiration from irrigated areas, therefore, is 4.3 ft. in depth, or 18 sec-ft., of continuous flow.

The bases for computing the evaporation and transpiration loss from grass and alkali land are the soil-evaporation equations of Fig. 10 and the ground-water contours of Plate VI. The equations were developed for a fixed ground-water surface, but they cover the periods from October 1st to March 31st and from April 1st to September 30th, which practically coincide with the observed periods of rising and falling ground-water. Hence, to cover the natural conditions of fluctuating ground-water surface, average annual depth to groundwater at a given point may be substituted in the equations instead of fixed ground-water depths. The average annual depth to groundwater, in feet, and the depth of evaporation, as determined from the equations for 1911, are given in Table 15 for the non-irrigated areas enclosed by the 3-ft. contour and between the 3- and 4-ft. and 4- and 8-ft. contours. The volume annually evaporating from the whole area enclosed by these contours is equivalent to a continuous flow for the year of 109 sec-ft.

TABLE 15.—TOTAL EVAPORATION FROM GRASS AND ALKALI LANDS IN THE VALLEY FLOOR.

(BASED ON 1911 DATA.)

Enclosing contours.	Area, in square miles.	Average depth to ground-water, in feet.	ANNUAL DEPTH OF EVAPORATION, IN INCHES.			Equivalent flow, in second-feet.
			Summer.	Winter.	Total.	
3 Ft.........	11.89	2.5	36.5	5.2	41.7	36.6
3-4 Ft.........	17.66	3.5	29.6	4.0	33.6	43.7
4-8 Ft.........	25.04	5.5	15.6	0.2	15.8	29.1
Totals.......	54.59	....	....	....	....	109.4

The water of Blackrock and Hines Springs, and of the small springs along the upper edge of the valley floor, spreads out in many shallow lake basins before reaching Owens River. The loss by evaporation from the surface of these lakes is large. Estimates based on the area of water surface exposed and the evaporation from water in the shallow pan in soil indicate that about 50% of the flow of these springs thus escapes into the atmosphere. As the combined flow is 31 sec-ft., the loss by evaporation from the free water surface is 15 sec-ft. The portion of the remainder which does not flow into Owens River percolates into the soil and escapes by evaporation from the soil and by transpiration.

Two springs derive their waters from percolation and discharge directly into Owens River; these are Upper and Lower Seeley Springs. Their combined average flow is 11 sec-ft. In addition, the Blackrock Springs discharge an average of 7 sec-ft. into the river during November to March, inclusive, which is equivalent to a continuous flow of 3 sec-ft. The total discharge into the river from springs, therefore, is 14 sec-ft.

The grand total ground-water losses, therefore, are 156 annual sec-ft., of which which 127 sec-ft. or 81% is by soil evaporation and transpiration.

RATE OF RECHARGE BY PERCOLATION.

From Precipitation.—All portions of the region receive precipitation, but there is wide local variation in the quantities which enter the ground and percolate downward to the surface of saturation. The impervious rock surfaces of the high mountain drainage areas shed

all precipitation which they receive except that lost by evaporation, but accretions to the ground-water from precipitation on the remaining areas of the region are of considerable importance.

Conditions are exceptionally favorable for percolation on the intermediate mountain slopes. As has been stated, the formation is very porous, and practically none of the run-off reaches living streams. All precipitation but that lost by evaporation, therefore, can be considered as percolating downward to the surface of saturation and becoming a permanent addition to the ground-water supply. Snow is practically all melted before May 15th, so that the period of direct exposure to evaporation is not as long as at higher levels, although the rate is greater. Evaporation losses from the moist soil are very small. Before it is melted, the snow blanket protects the soil surface, and by its gradual and uninterrupted melting it fills the capillary spaces to considerable depth, so that gravity water passes downward rapidly. When the snow disappears the rapid drying of the soil surface soon interrupts upward capillary movement, thus preventing further evaporation loss and allowing the percolating water to reach the surface of saturation. In view of these facts, the percolation factor is regarded as being about 0.75 for the more elevated areas receiving approximately 20 in. of precipitation. Less favored areas were assigned smaller factors after a study of their individual characteristics.

The results of computations of the total quantity of percolating water yielded by the intermediate mountain slopes are shown in Table 16. The method was to determine the mean seasonal precipitation at the center of area of each triangular subdivision, multiply this by the area in square miles, and apply a percolation factor. The area of each subdivision and the horizontal and vertical position of its center were obtained from Table 3. Diagrams 7, 8, 10, 11, 13, and 14 on Plate III were used in determining the depth of precipitation. The values differed slightly, as read from the altitude and distance diagrams, and the average was adopted as the most reliable. The total volume of precipitation on the 29.4 sq. miles of intermediate mountain slope is 27 580 acre-ft., of which 19 700 acre-ft. is a permanent addition to the underground water supply of the region. Expressed as a continuous flow, the total percolation from this area is 27 sec-ft.

The outwash slopes yield to the underground supply a much smaller volume of water, which is derived principally from slopes above

TABLE 16.—Percolation From Precipitation Upon Intermediate Mountain Slopes of Independence Region.

(Mean seasonal values.)

Taboose Group of Precipitation Gauges.

Adjoining high mountain drainage area.	Depth of Precipitation on Center of Area, in Inches.*			Volume of precipitation on area, in acre-feet.	Percolation factor.	Amount of Percolation.	
	A.	B.	Average.			Volume, in acre-feet.	Discharge, in second-feet.
Tinemaha	22.2	17.8	20.0	2 320	0.75	1 740	2.4
Red Mountain	23.6	17.8	20.7	2 620	0.75	1 970	2.7
Taboose	22.2	16.5	19.4	4 080	0.75	3 060	4.2
Goodale	18.5	20.0	19.2	2 350	0.75	1 760	2.4
Division	19.9	15.5	17.7	900	0.70	630	0.9
	....	....	....	12 270	....	9 160	12.6
Oak Group of Precipitation Gauges.							
Sawmill	18.2	18.4	18.3	1 290	0.70	900	1.2
Thibaut, North Fork	18.2	18.9	18.6	530	0.70	370	0.5
Thibaut, South Fork	14.5	15.0	14.8	60	0.60	40	0.1
Oak, North Fork	16.4	14.2	15.3	2 950	0.60	1 770	2.4
Oak, South Fork	16.4	15.8	16.1	880	0·70	620	0.9
Little Pine	17.7	15.8	16.8	1 810	0.70	1 270	1.8
Pinyon	18.7	20.8	19.8	3 050	0.75	2 290	3.2
	....	....	....	10 570	....	7 260	10.1
Bairs Group of Precipitation Gauges.							
Symmes	13.4	17.0	15.2	340	0.70	240	0.3
Shepard	12.7	12.4	12.6	650	0.65	420	0.6
Bairs, North Fork	12.4	10.8	11.6	300	0.65	200	0.3
Bairs, South Fork	14.8	12.0	13.4	860	0.70	600	0.8
George	15.8	15.8	15.8	1 760	0.70	1 230	1.7
Hogback	15.2	13.6	14.4	830	0.70	580	0.8
	....	....	....	4 740	....	3 270	4.5
Grand total	....	....	....	27 580	....	19 690	27.2

* Depth of precipitation as obtained by the precipitation-altitude diagram is given under A; as obtained by the precipitation-distance diagram under B. The average is taken for use in computations.

the 5 500-ft. contour. Precipitation occurs as snow less often here than on the higher slopes, and usually melts within a few days after falling. The capillary water in the upper layers of the soil thus has opportunity to evaporate after each storm, and it is only when several storms occur in succession that there is enough percolating water to

penetrate the ground beyond possibility of return. The long dry summer and the desert conditions draw all moisture from the ground to considerable depths, and the progress of percolating waters is slow because the capillary spaces must be refilled. Test pits dug in the region of the 4 500-ft. contour, 10 days after a series of storms, showed a penetration of capillary water to a depth of 4 ft. and the entire absence of gravity water. The total precipitation from these storms at this point was about 3.5 in., which, with a 28% available pore space, would represent 1 ft. of completely saturated soil. Considering the evaporation losses, it is not surprising that there was no gravity water within the depth of penetration observed. Observations made at higher elevations after this storm showed gravity water in considerable quantity at a depth of 12 ft. Percolation factors varying from zero to 0.60 were assigned to the several zones of the outwash slope as a result of these field observations.

The results of computations for the total quantity of percolating water yielded by the outwash slopes are shown in Table 17. The whole area of 165 sq. miles was divided into zones lying between contours at 500-ft. intervals from about 4 000 to 6 500 ft., and the zones in turn were divided into groups corresponding with the precipitation gauges. The method of computation was to average the precipitations for adjacent contours obtained from Diagrams 7, 8, 10, 11, 13, and 14 of Plate III. These averages represented the average precipitation for each zone in each group and, when multiplied by the area and the percolation factor, gave the quantity of percolating water which reached the permanent ground-water level. The total annual precipitation on the outwash slopes is 62 000 acre-ft., of which 16%, or 9 800 acre-ft., is effective percolating water. Expressed as a continuous flow, the volume of percolating water amounts to 13.4 sec-ft.

Throughout the valley floor the surface of saturation is so close to the ground surface that capillary connection is maintained during most of the year, and percolation from precipitation is rapid. The depth of penetration is usually slight, however, because precipitation in single storms is small. Several storms in succession or a warm rain on snow will result in a rise of ground-water, but the total average ground-water supply from this source does not exceed 4 sec-ft.

Direct percolation from precipitation, therefore, furnishes a grand total of 44 annual sec-ft. to the underground supply of the basin.

TABLE 17.—PERCOLATION FROM PRECIPITATION UPON OUTWASH SLOPES OF THE INDEPENDENCE REGION.

(Mean seasonal values.)

TABOOSE GROUP OF PRECIPITATION GAUGES.

Contours bounding precipitation zones.	Area of zones, in square miles.	DEPTH OF PRECIPITATION, IN INCHES.		Volume of precipitation on zone, in acre-feet.	Percolation factor.	QUANTITY OF PERCOLATION.	
		On contours.	On zone.			Volume, in acre-feet.	Discharge, in second-feet.
Grass–4 500..........	28.07	6.0, 7.7	6.8	10 180	0.00	0	0
4 500–5 000..........	8.54	7.7, 9.0	8.4	3 830	0.10	380	0.5
5 000–5 500..........	8.14	9.0. 10.8	9.9	4 300	0.20	860	1.2
5 500–6 000..........	5.56	10.8, 12.9	11.8	3 500	0.35	1 220	1.7
6 000–6 500..........	3.42	12.9, 15.2	14.0	2 550	0.60	1 530	2.1
	53.73	....	....	24 360	....	3 990	5.5
OAK GROUP OF PRECIPITATION GAUGES.							
Grass–4 500..........	24.57	4.8, 5.9	5.4	7 080	0	0	0
4 500–5 000..........	11.78	5.9, 7.3	6.6	4 150	0.05	210	0.3
5 000–5 500..........	9.23	7.3, 9.1	8.2	4 040	0.15	610	0.8
5 500–6 000..........	5.82	9.1, 11.3	10.2	3 170	0.30	950	1.3
6 000–6 500..........	4.82	11.3, 13.6	12.4	3 190	0.50	1 600	2.2
	56.22	....	....	21 630	....	3 370	4.6
BAIRS GROUP OF PRECIPITATION GAUGES.							
Grass–4 500..........	19.56	3.3, 4.0	3.6	3 760	0	0	0
4 500–5 000..........	11.68	4.0, 5.2	4.6	2 860	0	0	0
5 000–5 500..........	9.76	5.2, 6.9	6.0	3 120	0.10	310	0.4
5 500–6 000..........	9.23	6.9, 8.6	7.8	3 840	0.25	960	1.3
6 000–6 500..........	5.11	8.6, 10.3	9.4	2 560	0.45	1 150	1.6
	55.34	....	....	16 140	....	2 420	3.3
Grand total.....	165.29	....	....	62 130	....	9 780	13.4

From Stream Channels.—The most important source of underground water in a desert region is percolation from stream channels. This process is continuous from perennial streams, although it varies with the discharge of the streams and the temperature, as previously indicated. Beneath each stream channel as it crosses the outwash slope is a "ridge" of ground-water rising from the general plane of satura-

tion. The inclination of the slopes of this ridge and the breadth of its base vary periodically with the stage of the creek and the time of year. There is complete saturation within its slopes and a movement of gravity water toward the general ground-water surface. A considerable quantity of water also percolates from intermittent streams.

Percolation from stream channels in the Independence Basin is confined to the creeks draining mountain canyons. There are 17 of of these streams, 11 of which are perennial throughout their channels, 5 are perennial over the upper portion of their channels only, and 1 (Dry Canyon) is entirely an underground stream. The surface flow of the two most northerly of these streams, Tinemaha and Red Mountain Creeks, discharges northward across the Poverty Hills into the Bishop-Big Pine region, but the percolation from their channels is tributary to the Independence Basin. The channels of streams entirely within the basin are continuous from their canyons to the U. S. Geological Survey gauging stations, below which they divide, irrigation ditches carrying all the flow except during the high-water period of wet years, when the excess passes down the natural channels. The problem is thus divided into the determination of percolation above and below the Government gauging stations. The first subject has already been discussed at length and need not be considered here in detail. An inspection of Tables 4 and 5 shows that for the creeks from Taboose to Hogback, inclusive, the total 21-year average discharge at the mouths of the canyons is 130 sec-ft. and at the Government gauging stations 84 sec-ft. If the flow of 2 sec-ft. from Spring No. 2 on Division Creek is included with the canyon discharge, the percolation loss above the Government gauging stations is 48 annual sec-ft. To this should be added 6 sec-ft., as indicated by the diagrams, for Tinemaha and Red Mountain Creeks, making a total of 54 sec-ft.

The quantity lost below these stations is not so easily determined, on account of the numerous channels and irregular flow. Estimates were made on each creek, based on the length of main channel and distributing ditches outside of irrigated areas. The loss per mile was assumed to be the average annual loss per mile for the upper channel of the creek, and the total percolation loss from stream channels below the Government stations was estimated at 25 sec-ft. of continuous flow. This estimate does not include percolation from waste irrigation water or surplus creek water which has passed east of the ranches.

The grand total addition to the underground supply derived by percolation from stream channels, therefore, is 79 annual sec-ft.

From Irrigation.—Irrigation has been practised throughout this region, in connection with farming, for at least 30 years, and is a permanent factor in the underground water problem. The total area under systematic irrigation is approximately 3 000 acres, divided into a number of isolated ranch groups which depend on the mountain creeks for their supply. Oak Creek, the largest of these streams, supplies about 45% of the whole area. The remaining area is divided among eight creeks and the Stevens ditch, which during the period of observation has been largely supplied by the surplus flow of the creeks. The acreage irrigated from each source is given in Table 18. About 50% of this land was originally desert, lying along the lower margin of the outwash slope, and is very porous. The remainder lies in the valley floor, where permanent ground-water is within reach of plant roots and where clay soils predominate. The location of the several areas is shown on Plate VI. Alfalfa and grain are irrigated by

TABLE 18.—Estimated Net Volume of Water Used for Irrigation in the Independence Region During 1909.

Source of supply.	Area irrigated in acres. *	Duty of water per acre for season, acre-feet. †	Total Volume of Water Used.	
			Acre-feet.	Second-feet for 6 months.
Taboose Creek	(170)	(12)	2 040	5.6
Goodale Creek	(110)	(16)	1 760	4.9
Division Creek	(80)	(16)	1 280	3.5
Sawmill Creek	(90)	(16)	1 440	4.0
Oak Creek, ranch No. 1	109	7.22	790	2.2
Oak Creek, ranch No. 2	49	15.40	758	2.1
Oak Creek, ranch No. 3	155	2.80	435	1.2
Oak Creek, ranch No. 4	260	2.34	609	1.7
Oak Creek, ranch No. 5	38	16.40	623	1.7
Oak Creek	(80)	(16)	1 280	3.5
Oak Creek	(100)	(5)	500	1.4
Oak Creek	(560)	(3)	1 680	4.6
Little Pine Creek	(300)	(14)	4 200	11.6
Symmes Creek	(160)	(5)	800	2.2
Shepard Creek	(280)	(12)	3 360	9.3
George Creek	(160)	(12)	1 920	5.3
Stevens ditch	(310)	(8)	2 480	6.9
	3 011	....	25 955	71.7

* Areas in parentheses obtained from approximate field observations; other areas obtained by careful field measurement.

† Figures in parentheses assumed from observations on Oak Creek ranches.

flooding, and corn by the furrow method. Three crops of alfalfa are raised each year, and the irrigating season extends from about April 15th to October 15th, although some farmers irrigate 9 months in the year. Grain is irrigated early in the season, and corn late, so that the water is continually used. In most places the use of water is lavish, and no attempt is made to economize it or even to apply the quantity best suited to the crop and soil conditions.

A basis for determining the percolation from irrigation is a knowledge of the duty of water, or the quantity of water used in maturing a given area of crop. This was obtained in 1909 by carefully measuring the quantity of water used daily during the irrigating season on five typical ranches which derived their supply from Oak Creek. On ranches where there was a continual waste from irrigation the surplus water was also measured. Areas in crop were obtained from a careful stadia survey of each ranch.

With conditions on these typical ranches in mind, an examination was made of all other ranches in the region, and values were estimated for the duty of water on each. The number of acres irrigated and in crop was also determined approximately by reference to subdivisions of the public survey. From these data the volume of water used for irrigation was determined, as shown in Table 18. The total volume used during the 6 months, April 15th to October 15th, is about 26 000 acre-ft., equivalent to a continuous flow of 72 sec-ft. during the period. When spread out over 3 010 acres, this represents an average depth of 8.6 ft. This result probably represents an average practice throughout the Owens Valley, for the duty of water measured by the Reclamation Service during the season of 1904 on two typical ranches near Bishop was 7.11 and 9.17 acre-ft. per acre.

The distribution of this water, as regards evaporation, transpiration, and percolation beyond the reach of plant roots, is the next step in computing the ground-water supply from this source. Direct evaporation is relatively small, for the water when spread out over the fields is shaded by the crop and sinks rapidly into the ground. Probably 10% would cover this loss. The transpiration loss from alfalfa during the irrigation season has already been computed as 3.43 ft. depth of equivalent water, or 40% of the average volume of water applied to crops. The transpiration loss from corn and small grains is probably less in this locality, but the direct evaporation loss is greater. There-

fore 50%, or 4.3 ft. depth, represents the quantity of water applied in irrigation in this region which is absorbed by the atmosphere. The other 50% is a permanent addition to the ground-water supply, and is equivalent to a continuous flow of 18 sec-ft. throughout the year.

From Flood Water.—The quantity of percolation from surplus creek water, which spreads out over the valley floor to a greater or less extent, is difficult to determine. Of the 84 sec-ft. average flow at Government gauging stations, 61 sec-ft. are disposed of in channel percolation and irrigation. Possibly 5 sec-ft. of the remainder reach Owens River. This leaves 18 sec-ft. to be divided between evaporation and percolation in the flats between the ranches and the river. The area flooded averages about 5 sq. miles during June and July. The loss by evaporation during this period from a shallow pan in soil was about 24.5 in., and, as the conditions are similar, this represents approximately the loss from shallow flood water. The volume expressed as a continuous flow for two months is 55 sec-ft., or, for a year, 9 sec-ft. The other 9 sec-ft. can be assumed to represent the percolation from this flood water. It is not a permanent addition to the ground-water supply, however, for the surface of saturation is only a few feet below the ground surface in this area, and evaporation from damp soil and transpiration from natural vegetation soon reduce the ground-water surface to its normal position.

Summary of Percolation.—The four sources of ground-water are percolation from direct precipitation, from stream flow, from irrigation, and from flood water in the valley floor.

The first of these yields about 44 annual sec-ft., of which 61% is from the intermediate mountain slopes, 30% from the outwash slopes, and 9% from the valley floor. Percolation from streams yields about 79 annual sec-ft., of which 68% is above Government gauging stations and 32% below. Irrigation yields 18 annual sec-ft. and flood waters in the valley floor 9 annual sec-ft.

The grand total ground-water, therefore, is 150 annual sec-ft., of which probably 75% reaches the deeper strata of the valley fill. The rate of recharge of the basin, as thus determined, differs by less than 4% from the ground-water loss previously computed. The reliability of the data is thus confirmed as well as the correctness of the assumptions.

Safe Yield.

Thus far, this paper has presented conditions as they are found to exist in a natural state. The problems which the engineer has to solve are those connected with the artificial extraction of water from underground reservoirs. First among these is the determination of the safe annual yield or the limit to the quantity of water which can be withdrawn regularly and permanently without dangerous depletion of the storage reserve. A second problem which naturally accompanies the first is the devising of methods for increasing artificially the safe annual yield of reservoirs which are apparently already developed to the limit. The writer will outline his ideas, in the hope that they will suggest a constructive line of discussion which will lead to a better understanding of these subjects.

It is obvious that water permanently extracted from an underground reservoir, by wells or other means, reduces by an equal quantity the volume of water passing from the basin by way of natural channels. This is illustrated by the commonly recognized fact of the drying up of springs and cienagas as the result of heavy pumping. The theoretical limit for safe draft, exclusive of return water, therefore, is the average rate of recharge for a basin. The practical limit, however, depends on the relation of draft to storage capacity, within economic pumping limits. Where the storage capacity is very large as compared with annual draft, the theoretical and practical limits should nearly agree, as the storage reserve can be drawn on in periods of protracted drought. For basins with comparatively small storage capacity, the practical limit will be less than the theoretical. Draft computations may be made with the mass-diagram as ordinarily used in surface storage problems. Storage capacity is determined from the area of water-bearing material, limiting depth for economic pumping, and percentage of voids capable of depletion. The supply is the quantity of water annually absorbed by the porous material of the basin. This may be determined each year by methods similar to those used in the Owens Valley studies.

The draft thus obtained, however, is not the safe yield of the basin, for there are always certain residual losses which cannot be entirely prevented, such for instance as soil evaporation from cienaga lands. These residual losses must be ascertained and deducted from the gross draft. The quantity thus obtained may be persistently with-

drawn from the basin without causing general depression of the water plane to the point where pumping operations must cease for economic reasons.

The determination of residual losses presents difficult problems. Some of the conditions which are responsible for these losses are the following:

1.—The elevation of the impervious rim at the outlet being less than the elevation of the water plane in the lowest depression of the basin, thus allowing ground-water to escape as underflow. The quantity of water thus dissipated depends on the transmission capacity and area of the porous material overlying the rim and the available head. In most cases the volume of water thus lost is relatively very small.

2.—The outlet of springs being at a considerably lower elevation than the general water plane of the basin in the outlet region. Such a condition may exist where an arroya has cut a channel through the impervious rim at a point where the surface falls away rapidly down stream. Such losses are also relatively small.

3.—The occurence of water under Artesian pressure in underlying strata of porous material confined between more impervious layers of fine sand or clay. This is the least recognized, but yet the most important, cause of residual ground-water loss. The effect of Artesian pressure is to force moisture through pores or fractures in the impervious capping and thus maintain a permanent ground-water plane near the surface. The water continually supplied from below is disposed of either by evaporation from the soil surface and vegetation or by escaping at the surface in springs or seepages. These losses persist as long as the Artesian pressure is sufficient to force the water through the overlying strata. The volume of these losses during any period of one or more years bears a functional relation to the average Artesian pressure during the same period. The writer states these conclusions as the result of a careful study of records and conditions in a number of differently situated Artesian basins.

These conclusions are well illustrated by the following facts of common knowledge. First, consider the result of abnormally large precipitation for a period of one or more seasons. The ground-water accretions exceed the losses from the basin, the excess water accumulates in the voids of the porous gravels surrounding the confined strata,

and the free ground-water surface rises throughout the basin. Within the area of confined gravels hydrostatic or Artesian pressure increases. A greater quantity of water is forced through the overlying strata, not only over the area already moist, but from a circumscribing zone within which the pressure was previously insufficient to maintain a shallow ground-water surface. The observed result, therefore, is increased spring and seepage flow and an enlarged area of moist cienaga land from which evaporation occurs. Second, assume a series of dry years. Ground-water storage is depleted, the free ground-water plane falls, Artesian pressure decreases, and less water is forced through the overlying strata. The observed result is decreased spring and seepage flow and shrunken evaporating area. The latter occurs, because, for the new conditions, the rate of evaporation from the outer zone of moist soil exceeds the rate of supply from below. The accumulation of water in the soil is drawn on, lowering the water level to the limit of capillary action. A similar result occurs where relief is afforded to Artesian pressure by the drilling of many deep wells drawing from Artesian strata. In some of the Southern California Artesian basins, which formerly possessed cienaga lands, relief of Artesian pressure by wells and heavy pumping has dried up such lands.

The importance of residual losses, due to Artesian pressure, is forcibly shown by the Owens Valley studies, where it was found that in a natural state 81% of the total yield of the Independence Basin was lost by evaporation from soil and vegetation. Similar conditions, in a slightly less degree, existed in many of the Southern California basins before ground-water development was undertaken. The increased pumping of the last 10 or 15 years has eliminated evaporation losses almost entirely in some of the smaller basins. In the larger basins, such as the San Bernardino Valley and Coastal Plain, the reduction has not been as great, having ranged from 30 to 50% of original evaporation losses in the various basins. In these basins evaporation losses formerly represented from 50 to 75% of the total ground-water supply. Hence, the residual evaporation losses to-day represent from 15 to 35% of the total ground-water supply for the large Southern California basins, and can be said to average 25 per cent.

The quantitative determination of the residual losses from an underground reservoir can only be made after a detailed study of the local

conditions. The factors to be considered are the topography and geology of the basin and its porous filling, the distribution and type of sources of percolating water, the rate of evaporation and transpiration, the depth of capillary action and the character of the soil within the evaporating area, the necessity for irrigation and the value of overlying lands for agricultural crops, the present or probable ultimate method of development of water, and the present or probable future use. The general condition favorable for small residual losses is the possibility of eliminating the evaporating area by lowering the water plane below the reach of capillary action. This may be done by the relief of Artesian pressure, by shallow pumping within the evaporating area, or by drainage. The first of these methods would result in reduced pressures in existing wells in the Artesian area and possibly lowered water levels in the back-water zone, especially if the ground surface has a steep slope. Shallow pumping and drainage, on the other hand, may be physically impractical or prohibitive in cost. Their success depends largely on the existence of shallow water-bearing strata from which water can be readily drawn.

There are three cases which arise in the determination of residual losses: first, a basin already fully developed or suffering from overdraft; second, a basin where the supply is partly developed; third, a basin entirely undeveloped. The first case can be recognized by inspection of the present or former evaporating area in connection with local confirmatory evidence and past records of water levels, yield of wells, etc. The residual losses may be ascertained from observations and measurements of existing conditions. The second and third cases require assumptions as to the method or combination of methods by which residual losses will be reduced to a minimum. These assumptions should be made after a careful study of local physical conditions and the probable future use of the water. The next step is the determination of existing evaporation losses, by contouring the ground surface and water plane, and ascertaining the soil evaporation losses for various depths to ground-water. The final step, namely, the determination of the percentage by which existing losses will be reduced by future development, is as yet largely a matter of judgment. Having arrived at some definite value, however, the residual losses due to evaporation can be computed, and, when combined with the losses from underflow or deep springs, the total quantitative result is obtained.

The thorough investigation of residual losses is essential in any determination of safe yield, and for this reason the writer has discussed the subject in detail.

Passing now from the determination of safe yield as limited by existing conditions to a discussion of methods for increasing safe yield artificially. Obviously, to accomplish the latter, either the rate of recharge of the basin must be increased or the percentage of unused water escaping from the basin must be decreased. Practical methods which suggest themselves are:

(1) Reduction of residual losses to the lowest possible quantity;
(2) Elimination of needless waste of underground water; and
(3) Increased absorption of surface flood waters.

The subject of residual losses has already been discussed at length. The writer wishes to emphasize the fact, however, that a basin has not been fully developed as long as the evaporating area persists in years of drought. The evaporating area is fully as important a criterion of the relation of withdrawals to supply as the water plane. A falling water plane does not of itself indicate overdraft. It is only when a rapid shrinkage and disappearance of the evaporating area accompanies a falling water plane that dangerous overdraft is indicated. Hence, in a closed Artesian basin from which the evaporation area has not disappeared, a greater yield may be obtained, provided an intelligent plan of development is followed.

A very common source of needless waste is from Artesian wells which are allowed to flow when the water is not in use. The wastefulness of this practice is so evident that it need not be discussed in this paper. Its continuance is made possible by lack of recognition of the ultimate effect of the practice among the owners of such wells. The writer feels that it is every engineer's duty and privilege to assist in guiding aright public opinion in matters of common interest, and suggests the subject of conservation of Artesian water as being pertinent in many communities.

The practicability of increasing the ground-water supply by bringing about greater absorption of flood water has been demonstrated in a number of California basins. The most extensive work of this kind is probably that done on the alluvial cone of Santa Ana River in the San Bernardino Valley. The method there used is to divert the flood

water of Santa Ana River in contour ditches from which it is distributed into smaller ditches which in turn subdivide, until finally the water spreads out over the porous alluvial gravels in a thin sheet and is absorbed. During the past few years the volume of water thus stored has averaged 12 000 acre-ft. annually, costing about 15 cents per acre-ft., including interest on the cost of permanent works. The work is capable of further expansion. The ultimate limit will be the ability to handle the violent floods which are of frequent occurrence. The problem is not that alone of controlling the water, but of disposing of silt. The water is normally clear and is free from silt soon after the flood crest passes. Flood water, however, carries great quantities of silt, which deposits as soon as the velocity is checked. This forms an impervious layer of slime which seals the gravels and must be broken up and eventually removed in order to use the same spreading ground continuously.

The conditions on most California streams tributary to closed basins correspond with the Santa Ana. The writer is of the opinion that complete absorption of even ordinary floods on these streams cannot be brought about without temporary surface storage. This must be accomplished either by utilizing storage sites in the stream channel or by construction of contour levees on the alluvial cone. The purpose of such reservoirs would be to act both as settling basins and as temporary storage sites. From these reservoirs the clear water would be released and brought into contact with the absorbent gravels by any method which proved most efficient under the local conditions.

In conclusion it may be said, first, that the rate of recharge of underground reservoirs of the closed-basin type is a definite quantity capable of measurement with a fair degree of accuracy; second, that safe yield is a quantity less than the rate of recharge, its quantity depending on the available storage reserve of the basin and residual ground-water losses; third, that under certain circumstances it is possible to increase the safe yield of a fully developed basin.

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CINDER CONCRETE FLOORS.

By Guy B. Waite, M. Am. Soc. C. E.

To be Presented May 6th, 1914.

Summary.

In the following paper the manner of using reinforced cinder concrete floor slabs by various cities in the eastern part of the United States is described. Especial attention is called to the arbitrary manner of using this material, and the insistence of continuing this arbitrary use, by New York City, as indicated by the latest building codes.

Attention is called to the unit stresses adopted by various cities and to recent data tending to establish unit working stresses from actual tests. The qualifications of this material for the purposes described are discussed, and a comparison with stone concrete is made.

The manner of arbitrary testing is described, and the difference between the conditions of such tests and the actual loadings found in buildings is pointed out.

Analysis is made of the arbitrarily approved reinforced cinder concrete floor slabs, showing the stresses which would be imposed on the materials by the maximum approved loads.

Cinder concrete is described and tabulations are made for the location of the neutral axis and for the carrying capacities, assuming various unit stresses. The influence of the stresses due to an excess of steel or to an excess of concrete is discussed.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Analyses of stresses allowed by such tests indicate that in some cases approvals have been given for dangerous construction; it is pointed out that arbitrary tests can be made which indicate carrying capacities not possible under the conditions actually found in buildings; and it is shown by formulations and tabulations that reinforced cinder concrete may be easily and uniformly designed by adopting working unit stresses, and that several cities have adopted such stresses for this material.

The writer believes that in this paper he has amply demonstrated that the only manner in which this material can ever be uniformly used under various conditions is for each locality to adopt unit stresses suited to its own conditions.

Where cinders of good quality can be obtained, cinder concrete, reinforced with steel, for fire-proof floor construction, is used very extensively in the eastern part of the United States. It is estimated that more than 50% of the large fire-proof constructions of Philadelphia, Pa., have reinforced cinder concrete floor slabs, and New York City has probably a larger proportion.

Floor slabs of stone and gravel concrete, reinforced with steel, of the same general form as cinder concrete, have been systematized by engineers so that they are being used on a rational basis; but cinder concrete has been allowed to straggle along, without much attention from engineers in general, and is being used in a varied manner. Most large cities, such as Philadelphia, Pa., Boston, Mass., and Chicago, Ill., require cinder concrete for floor slabs to be designed, in thickness and in quantity of reinforcement, with certain values of unit stresses as a limit.

In New York City, and in most small cities of the eastern part of the United States, unit stresses are not considered in the use of such concrete. In these places, the use is either entirely arbitrary or a separate test is required for each condition.

In New York City, since the beginning of the use of cinder concrete floors, in 1896, there have been some 200 approvals for reinforced floor slabs of this material, based on load tests. These approvals were for varying spans, thicknesses of concrete, and quantities of reinforcement. The tests have generally been conducted with a uniformly

distributed load—of pig iron, bags of cement, or sand—imposed on the slab.

As will be shown in detail, the results of former tests, which include all principal constructions in use, are extremely variable. The writer has been informed that since 1911 concentrated load tests are required for all new approvals, by the Borough of Manhattan, New York City. The result is that now there are approvals of concrete floor slabs in which the effective depth is 3 in. (4 in. total thickness) and the reinforcement is less than ½ lb. per sq. ft. on an 8-ft. span, and with a live floor load of 250 lb. per sq. ft. In these slabs the concrete is in the proportion of 1 : 2 : 5.

One of the dealers in reinforcing materials for cinder concrete publishes a statement of what he calls its "authorized use" by the Boroughs of Greater New York City. This is a 1 : 2 : 4 mixture, with a 5-in. effective depth up to 12-ft. spans for live loads of 60 lb. per sq. ft.—or a total load of about 120 lb.

Each contractor in New York City is held strictly to the details of construction for his approvals. For instance, if the approval allows a live load of 250 lb. per sq. ft. up to 6-ft. spans, and it is desired to use the construction for a live load of only 60 lb. per sq. ft., on 6 ft. 6-in. spans, this construction will not be allowed for this excess of 6 in. in span, though stressed less than with the approved loads.

Moreover, stone and gravel reinforced concrete floor slabs are required to be designed according to the following values: Extreme fiber stress in concrete, 650 lb. per sq. in.; maximum tension in mild steel, 16 000 lb. per sq. in.; ratio of moduli, 15. According to these values, stone concrete slabs in some instances would be designed with a greater thickness and have greater reinforcement than the approvals for cinder concrete.

The writer believes that the excuse, that cinder concrete is so extremely variable as not to permit of the assignment of unit stresses, is not well founded. The fact that cinder concrete is being used extensively in engineering construction is sufficient reason for assigning, for safety, some unit stresses, in order to insure uniformity. The present manner of making arbitrary approvals has led to confusion, various forms of reinforcement obtaining approvals in which the spans, thicknesses of concrete, quantities of reinforcement, and floor loads vary beyond description.

The proposed building codes of 1912 and 1913 have persistently continued the present manner of using cinder concrete (with slight modifications) for floor constructions.* In these codes, the manner of designing and constructing cinder concrete floor slabs is arbitrarily specified. In the proposed code of 1913, the allowed carrying capacity of the slab with 3 in. effective depth on 8-ft. spans is increased (over the 1912 code) to a live load of 250 lb. per sq. ft.

It is on account of these conditions in the uses of cinder concrete that the writer has been prompted to bring this matter before the Society, with the hope of interesting the Engineering Profession in a subject which should be of vital concern to many.

Cinders.—Cinders for concrete floor construction are understood to mean either hard or soft coal ashes coming from boiler plants. Coal ashes from a stove or a small house boiler are generally too finely burned to be of use in this kind of concrete. Cinders from either hard or soft coal, and having sharp particles greatly in excess of the smooth ashes, make a concrete of considerable strength and of good fire-resisting qualities.

Cinders vary in size from the consistency of coarse building sand to clinkers several inches in diameter. Some hard-coal cinders run so perfectly in grade that good concrete is made without the addition of sand; more often, however, sand must be added. The quantity of sand required to fill the voids is variable, but the usual specification is 2 parts of sand to 5 parts of cinders.

After a long experience in the use of cinder concrete, the writer believes that there should be no danger in assigning working stresses to the material, these stresses being taken from tests on known mixtures in which cinders of the lowest grade permitted are used. If the material is too poor to have such working stresses assigned, it seems evident that cinder concrete is not a safe material for such an important part of the structure as the floor slab.

Cinder Concrete as Fire-Proofing.—Since 1896 some 82 different floor constructions have been tested in the United States by fire and water. Nearly all these tests were made under the auspices of the Bureau of Buildings of New York City, on full-sized floor slabs. About 40 of the tests were on reinforced cinder concrete floor slabs, some 23

* Sec. 113, proposed code of 1912; and Sec. 104, proposed code of 1913.

on stone or gravel concrete floor slabs, 10 on some form of hollow tile, and the others on special floor constructions.

The test structures were about 14 ft. square and 9 ft. high, with a fire grate at the bottom and the floor slabs to be tested forming the ceiling. The average temperature on the tested floor slabs was 1 700° Fahr. maintained for 4 hours. Following this there was a water test of 60 lb. pressure through a 1⅛-in. nozzle applied for 10 min. to the under side of the slabs. During the fire test a load of 150 lb. per sq. ft. remained on the entire surface of the slabs. After the fire test the slabs were subjected to a total load of 600 lb. per. sq. ft. Fig. 1 is a view inside one of these test structures after a fire test.

In these tests, the cinder concrete floor constructions withstood the fire and water better than any other material. In the writer's opinion, the reason for this is not due to cinder concrete being more fire-proof than the others, but simply to the fact that it is not ruptured and destroyed by the expansions and contractions caused by fire and water.

Hollow tile, as a material, may be more fire-proof than cinder concrete, but the stresses due to the expansion of the exposed surface, destroy it.

The test structure, an inside view of which is shown, was built entirely of cinder concrete, mixed 1 to 4 without sand, and withstood four different fires before being torn down to make room for improvements. Only the surface, for a depth of less than 1 in., was affected; this was dehydrated, but the burned cinder concrete was still intact, and protected the remainder of the material. In this concrete there was at least 5% of unburned pea coal which was unaffected by the fire.

Arbitrary Testing of Cinder Concrete.—In New York City after each type of cinder concrete construction had qualified for fire-proofing, the variations in spans, carrying capacity, etc., were determined by each manufacturer by constructing the slabs he proposed to use and testing them with uniformly distributed loads under the supervision of the Bureau of Buildings.

This was the method used for the approvals of the constructions shown, with the exception of Fig. 5, which was constructed for fire test. These constructions were taken from the files of the Bureau of Buildings from among some 200 similar approvals.

For comparison with the approved cinder concrete floor slabs shown by Figs. 3 to 7, the writer gives the relative thicknesses and the quantity of reinforcement for slabs under similar conditions according to the unit stresses used by Philadelphia, and also the thicknesses and quantity of reinforcement required by New York City for 1 : 2 : 4 stone concrete.

It is to be noted, in comparing the stone or gravel concrete construction (Figs. 11 to 13) with the cinder concrete, that the former is made unquestionably continuous, and the latter, in most cases, is cut into and is simply supported by the steel beams (Figs. 3 to 10, and 14 and 15).

The approvals for the longest spans with the highest carrying capacity were based on tests with slabs which were continuous over supports (Figs. 3 to 7). In practice, it is seldom that cinder concrete floor slabs between steel beams can be made continuous throughout a floor.

In Figs. 14 and 15, which are assumptions for illustration, it will be noticed that the reinforcement is simply laid over the top flanges of the supporting steel beams. In practice, there is no way of taking up the slack in the reinforcement coming over the tops of the beams, and the consequences are that under such conditions there can be no continuous action—such as engineers understand—in the combined steel and concrete structure. Unless considerable slack is allowed on the sides of each supporting beam, the tamping of the concrete works the reinforcement to the top of the slab.

In buildings where a wooden floor is used, the sleepers are generally laid directly on the steel beams, and consequently the concrete is kept below the top flanges of the beams. Here, therefore, the concrete is not continuous in any part of the floor; but in the interior spaces on all floors, whether continuous or not, the channels on the outside spans, and against openings in floors, cannot have either concrete or reinforcement continuous over them (Figs. 14 and 15).

To any one acquainted with the actual construction of cinder concrete floors, it must be apparent that some parts of each floor will not admit of being constructed with the same care that might be attained in a test construction; therefore it does not follow that because a slab was continuous in the test, it is so in actual practice.

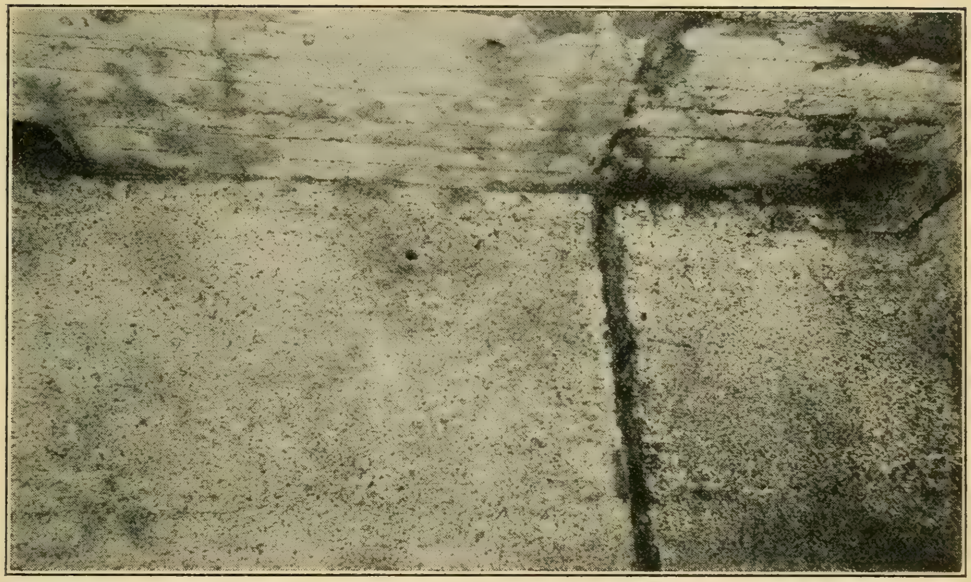
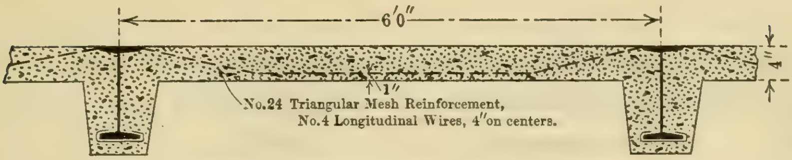


FIG. 1.—CEILING AND SIDE OF CINDER CONCRETE, AFTER THE SECOND FIRE. PLASTER ON PART OF CEILING.

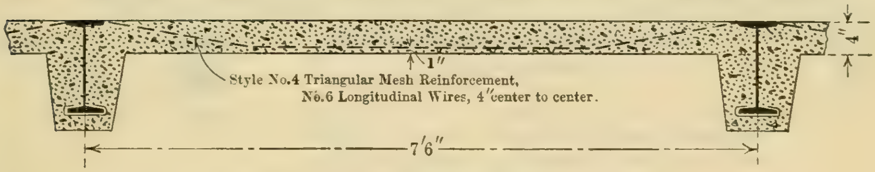
FIG. 2.—ARBITRARY TEST LOAD. SAND IN BAGS ON 4-IN. SLAB, 36 IN. WIDE AND 6 FT. LONG.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 400 lb. per Sq. Ft.

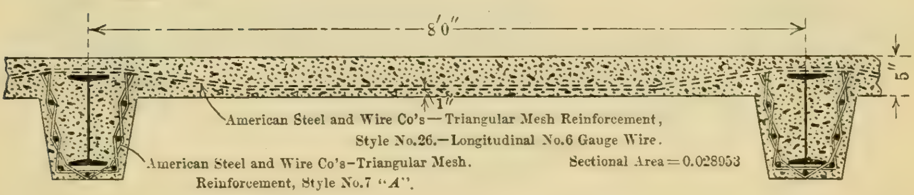
FIG. 3.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 95 lb. per Sq. Ft.

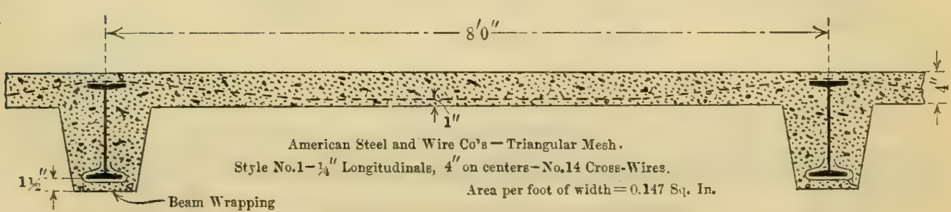
FIG. 4.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 150 lb. per Sq. Ft.

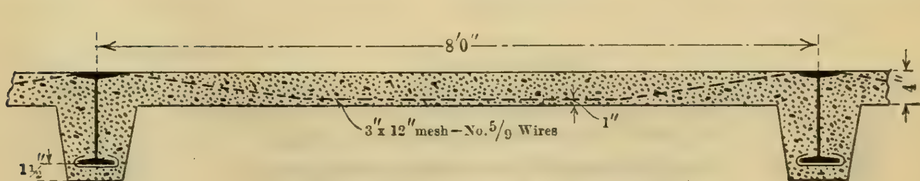
FIG. 5.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

Working Load: 200 lb. per Sq. Ft.

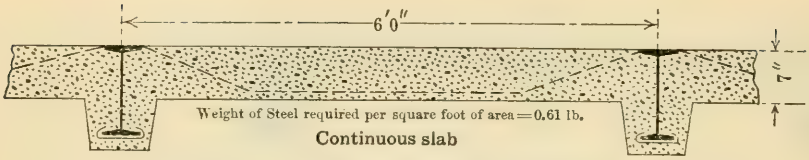
FIG. 6.



Concrete: 1 Portland Cement, 2 Sand, 5 Cinders.

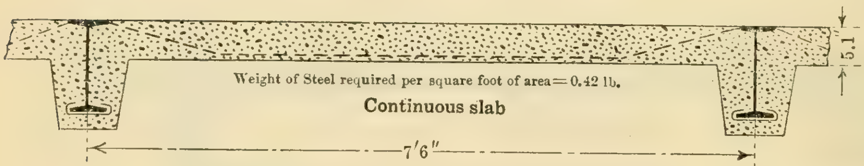
Working Load: 250 lb. per Sq. Ft.

FIG. 7.



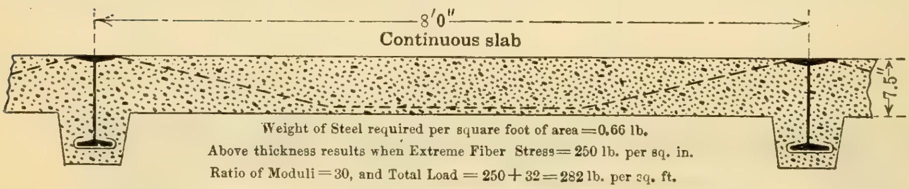
Above thickness results when Extreme Fiber Stress = 250 lb. per sq. in.
Ratio of Moduli = 30, and Total Load = 400 + 32 = 432 lb. per sq. ft.

CINDER CONCRETE.
FIG. 8.

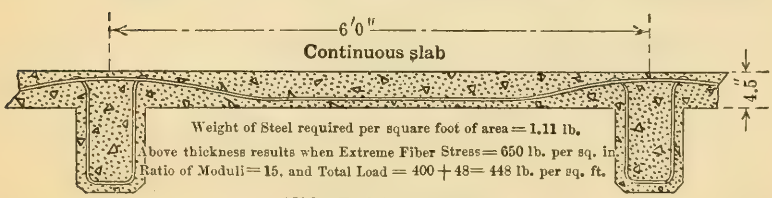


Above thickness results when Extreme Fiber Stress = 250 lb. per sq. in.
Ratio of Moduli = 30, and Total Load = 95 + 32 = 127 lb. per sq. ft.

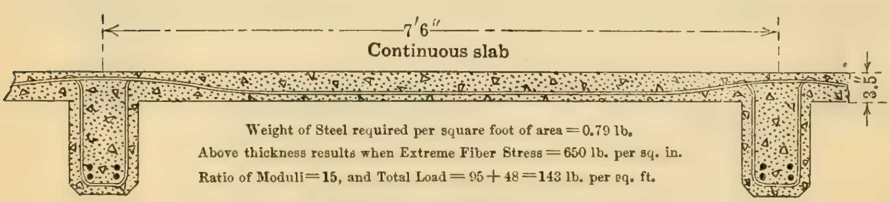
CINDER CONCRETE.
FIG. 9.



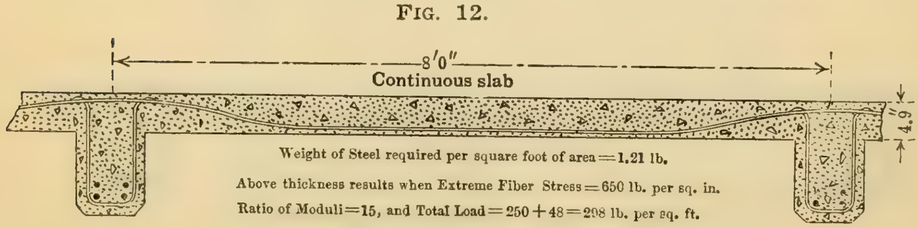
CINDER CONCRETE.
FIG. 10.



STONE CONCRETE.
FIG. 11.



STONE CONCRETE
FIG. 12.



STONE CONCRETE.
FIG. 13.

Again, these tested slabs were held by adjacent slabs on each side of the supports, thus enabling each side to take thrust. Now, in practice, outside slabs next to walls, elevator shafts, stairways, show windows, etc., are only resisted for thrust by tie-rods and by concrete slabs on one side.

If the writer is correct in his statement that outside slabs, etc., are not under the same conditions as the tested ones, where the reinforcement and the concrete is carefully made continuous, then this form of so-called continuous construction is being used in an unapproved and unsafe manner.

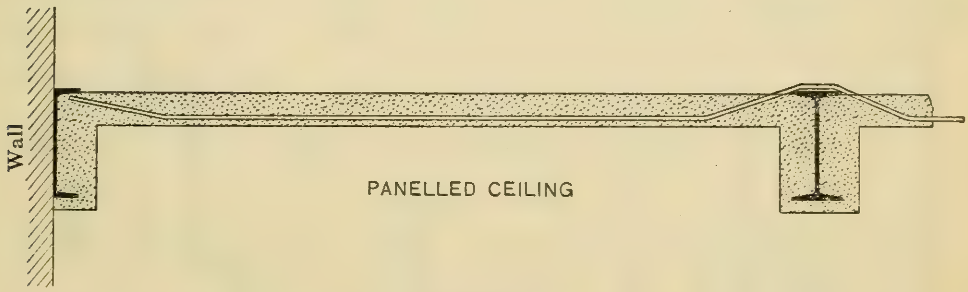


FIG. 14.

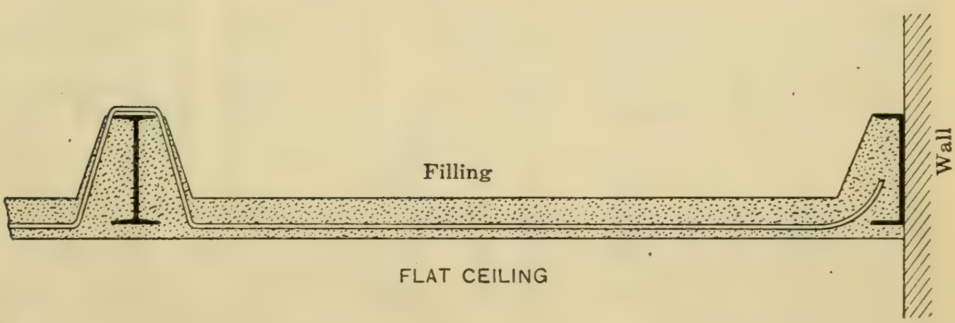


FIG. 15.

The tests on which the approvals were based had the kind of loadings heretofore mentioned. Fig. 2 shows a test loading similar to those described.

With the conditions of continuous construction under which these slabs were tested, with possible arching of loading, and with supports braced against side flexure, the stress that came on the steel and concrete at the middle of the slab can only be guessed. Still, according to the latest proposed building code (that of 1913) tests and

TABLE 1.—REINFORCEMENT.

(From Proposed Building Code of 1913, Sec. 104, Art. 8.)

Tension Wires 4 Inches on Centers.

Live load per square foot of floor area.	Type of reinforcement.	Span Between Steel Beams.					
		6 ft. 0 in. or less.		6 ft. 1 in. to 7 ft. 0 in.		7 ft. 1 in. to 8 ft. 0 in.	
			(Area of steel per foot width of slab.)		(Area of steel per foot width of slab.)		(Area of steel per foot width of slab.)
100 lb. and less..........	Bars................	0.4 lb.	(0.118 sq. in.)	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)
	Wire mesh.........	No. 7 wire	(0.0735 sq. in.)	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)
101 to 150 lb.............	Bars................	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)	0.8 lb.	(0.236 sq. in.)
	Wire mesh.........	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)
151 to 200 lb.............	Bars................	0.7 lb.	(0.206 sq. in.)	0.9 lb.	(0.265 sq. in.)	1.1 lb.	(0.324 sq. in.)
	Wire mesh.........	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)
201 to 250 lb..........	Bars................	0.8 lb.	(0.236 sq. in.)	1.0 lb.	(0.294 sq. in.)	1.2 lb.	(0.353 sq. in.)
	Wire mesh.........	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)	No. 2 wire	(0.162 sq. in.)

Matter in parentheses added for comparison. All other matter as per code.

approvals of that kind were to have preference over unit stresses. The following is quoted from Sec. 104, Art. 8, of this proposed code:

"The concrete is made 1, 2, 5; slab thickness, 4 in., with reinforcement 1 in. from bottom.

"Reinforcement must be as per tabulations."

It will be noticed from Table 1 that when the reinforcement is of wire mesh the area required is only about one-half as great as for any other form of reinforcement—even with the same kind of wire but without the mesh.

Use of Cinder Concrete by Cities Other than New York.—The cities mentioned in Table 2 use a ratio of moduli of elasticity of about 30, and the highest allowed extreme fiber stress in any of these cities is 300 lb. per sq. in.

TABLE 2.—Unit Stresses for Cinder Concrete, as Used by City Departments.

City.	Extreme fiber stress, in pounds per square inch.	Ratio of moduli of elasticity.	Kind of cinders used.	Mixture.	Working stresses in steel, in pounds per square inch.	Preferred reinforcement.
Philadelphia, Pa.	250	30	Hard coal...	1 : 2 : 4	16 000	None.
Boston, Mass....	300	25	Soft coal....	1 : 2 : 4 to 1 : 2½ : 5	16 000	None.
Chicago, Ill.....	245	30	Hard or soft.	1 to 8 aggregate.	18 000	None.
Baltimore, Md...	300	30	Hard or soft.	1 : 2 : 4.	15 000	None.

The results of tests on cinder concrete quoted in Table 3 were published recently in a technical journal,* these being the first of a series conducted at Columbia University with the co-operation of the Bureau of Buildings of New York City.

Though the limited tests in Table 3 would seem to show that a ratio of moduli of less than 30 could be used, the writer has been informed that specimens "C" were from extra good concrete, and that "A" and "B" are nearer the average material. In this case the ratios used by Philadelphia, Chicago, and Baltimore seem to be about right. As will be seen later, a less ratio of moduli will give a less carrying capacity, other things remaining the same.

**Engineering News*, October 9th, 1913.

In the tests at Columbia University, just referred to, it is the feeling, at the present stage of these tests, that an extreme fiber stress of from 200 to 250 lb. per sq. in. on cinder concrete is all that can be used conservatively.

Only the tests up to 6 months old have been made; the final tests may alter this assumption, but, for discussion, these values are sufficiently correct.

TABLE 3.

"WEIGHT AND COMPRESSIVE STRENGTH OF CINDER CONCRETE AS USED IN FIREPROOF FLOORS, NEW YORK CITY.

(The table covers 120 samples. Each figure given is the average of ten samples.)

Zone.	A.	B.	B2.	C.
Mix	1:2:5	1:1:5	1:2:5	1:2:5
Weight, lb. per cu. ft	107	100	107	109
One-month Test:				
Crushing Strength, lb. per sq. in	407	507	818	980
Mod. of Elast., lb. per sq. in	924 600	857 400	1 230 000	1 492 000
Two-month Test:				
Crushing Strength, lb. per sq. in	701	662	1 254	1 035
Mod. of Elast., lb. per sq. in	1 134 000	1 030 000	1 740 000	1 428 250
Six-month test:				
Crushing Strength, lb. per sq. in	933	754	1 744	1 478
Mod. of Elast., lb. per sq. in	971 000	1 050 000	1 348 000	1 276 000

NOTE.—B was hand-mixed; A, B2, and C were machine-mixed. The modulus of elasticity was determined at a point of the elastic curve corresponding to one-fourth the ultimate strength."

Designing Reinforced Concrete Floor Slabs.—Solid slabs of cinder concrete are not weak in shear, so that the discussion of vertical and diagonal shear will not be necessary. In the writer's observation of many breaking tests on cinder concrete floor slabs, there has never been a failure near the supports; almost uniformly, the breaking has taken place near the center of the slab. Usually, the top of the central portion has spalled off (cup fashion), apparently due to horizontal shear and compression, after which the slab suddenly collapsed.

Adhesion.—The adhesion of concrete to various kinds of reinforcement is shown by Table 4, the results of a series of tests on 1:2:4 cinder concrete, made in 1904 by Messrs. H. B. Gaylord and H. A. Pratt, at Stevens Institute of Technology. The cement was Lehigh Portland, and the cinders were as found coming from a boiler plant. The concrete blocks were 6 by 6 in., and the steel was embedded in the center.

It may be noted from Table 4 that the plain bars held in adhesion quite as well as the deformed bars; the average adhesion of surface embedded being about 200 lb. per sq. in. This same fact, as to the relative adhesion of plain and deformed steel in stone concrete, has already been shown in numerous tests. Therefore, it will be considered that the form of the reinforcement is immaterial so long as it has sufficient surface embedded.

Bending.—Proceeding on the ordinary assumptions of the common theory of flexure, Fig. 16 is a stress diagram, at the right of which

TABLE 4.—RESULTS OF ADHESION TESTS ON 1:2:4 CINDER CONCRETE.

No. of specimen.	Length embedded, in inches.	Perimeter, in inches.	Area of cross-section, in square inches.	Shape of bar.	Kind of concrete.	Pull, in pounds per inch.	Pull, per inch of perimeter.	Pull per square inch.
1	6¼	1.75	0.094	De Mann.	Cinder, 1:2:4:	396	1 414	206.2
2	11⅜	1.75	0.094	"	"	598	3 885	247.2
3	6¼	1.75	0.094	Flat.	"	376	1 343	214.6
4	16	1.75	0.094	"	"	250	2 285	142.8
5	18	1.75	0.094	"	"	230	2 371	131.4
6	23	1.75	0.094	"	"	176	2 072	89.8
7	5	1.57	0.196	Round.	"	400	2 730	255.0
8	6⅛	1.57	0.196	"	"	553	2 076	339.2
9	9	1.57	0.196	"	"	647	3 710	412.2
10	13	1.57	0.196	"	"	379	5 460	196.0
11	16	1.57	0.196	"	"	417	4 331	277.0
12	28¾	1.57	0.196	"	"	239	4 188	145.0
13	6	1.625	0.156	½ by $\frac{5}{16}$-in.	"	500	1 834	307.6
14	10¼	1.625	0.156	"	"	146	914	90.0
15	12	1.625	0.156	"	"	500	3 668	308.0
16	17¼	1.625	0.156	"	"	345	3 661	212.3
17	19¾	1.625	0.156	"	"	310	3 778	191.0
18	27¼	1.625	0.156	"	"	232	3 895	143.0
19	4¾	3.25	0.625	1 by ⅝-in.	"	990	1 477	311.0
20	6¾	3.25	0.625	"	"	800	1 661	246.3
21	14¾	3.25	0.625	"	"	665	3 015	204.6
22	20¼	3.25	0.625	"	"	680	4 353	215.0
23	26¾	3.25	0.625	"	"	645	5 303	193.0
24	11½	3.125	0.203	1 by ⅜-in. channel	Stone.	895	3 168	247.9
25	18	3.125	0.203	"	"	610	3 505	194.7
26	22½	3.125	0.203	"	"	523	3 433	103.8
27	6⅜	3.125	0.203	"	Cinder.	840	1 715	273.1
28	8⅛	3.125	0.203	"	"	848	2 304	271.1
29	16¾	3.125	0.203	"	"	245	1 120	166.8
30	17	3.125	0.203	"	"	555	3 019	177.6
31	20¼	3.125	0.203	"	"	435	2 216	141.0
32	22⅛	3.125	0.203	"	"	476	3 435	154.0
33	35¾	3.125	0.203	"	"	294	3 357	93.9
34	6⅛	1.00	0.0625	Ransome	"	431	2 800	439.8
35	7½	1.00	0.0625	"	"	377	2 825	376.6
36	10	1.00	0.0625	"	"	409	4 085	408.5
37	13¼	1.00	0.0625	"	"	334	4 435	334.8
38	22	1.00	0.0625	"	"	270	5 250	238.6
39	26½	1.00	0.0625	"	"	755	2 000	75.4
40	6	1.00	0.0625	Square.	"	442	2 650	441.6
41	7¾	1.00	0.0625	"	"	322	2 500	332.5
42	11½	1.00	0.0625	"	"	322	2 500	217.4
43	15½	1.00	0.0625	"	"	261	3 200	206.4
44	25¼	1.00	0.0625	"	"	128	3 215	127.3
45	32¾	1.00	0.0625	"	"	113	3 650	111.1

is shown the strain diagram used in determining the location of the neutral axis. Fig. 17 is for illustration, and Fig. 18 is a heavy-load stress diagram for purposes of discussion.

Let E_c = the modulus of elasticity of cinder concrete;
" E_s = " " " " " mild steel;
" f_c = " extreme fiber stress on cinder concrete (working);
" f_s = " unit allowed stresses per square inch on mild steel;
" a = " area of reinforcing steel, in square inches, or in pounds per square foot;
" y_1 = " distance from top of slab to neutral axis, or
" y_2 = " " " neutral axis to center of gravity of steel;
" d = " distance from center of gravity of steel to top of slab;
" T = " total thickness of the slab; here taken as $d + 1$.

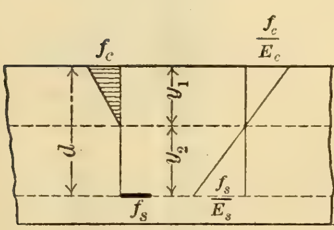


FIG. 16.

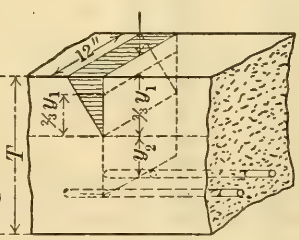


FIG. 17.

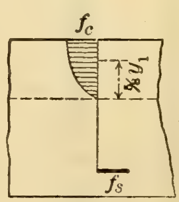


FIG. 18.

Referring to the diagram at the right of Fig. 16, it is evident that the maximum strain on the extreme fibers is $\frac{f_c}{E_c}$ and that the strain on the steel is $\frac{f_s}{E_s}$. Both these strains being the bases of two similar triangles, they are proportional to their altitudes, or $\frac{f_c}{E_c} : \frac{f_s}{E_s} = y_1 : y_2$; whence, $y_2 = y_1 \frac{f_s}{f_c} \times \frac{E_c}{E_s}$. If, $\frac{f_s}{f_c} = n$, and $\frac{E_c}{E_s} = m$, then $y_2 = n\,m\,y_1$; but, $d = y_2 + y_1$; hence,

$$y_1 = \frac{d}{n\,m + 1} \qquad \text{(1)}$$

Tabulation of Values for y_1.—As there may be differences of opinion as to the proper ratio of the moduli of cinder concrete and steel, the writer has prepared Table 5, giving values for y_1 (the dis-

tance from the top of the slab to the neutral axis) for different moduli, and for different ratios of unit stresses of extreme fiber on the concrete, and tension in the steel.

TABLE 5.—Distance from Extreme Fiber of Concrete to Neutral Axis, in Terms of d.

$\frac{E_s}{E_c}$ where $E_s = 30\,000\,000$ and E_c ranges from 800 000 to 2 500 000	y_1 when $\frac{f_s}{f_c} = 80$ *i. e.*, $\frac{16\,000}{200}$ or $\frac{20\,000}{250}$	y_1 when $\frac{f_s}{f_c} = 64$ *i. e.*, $\frac{16\,000}{250}$ or $\frac{20\,000}{312.5}$	y_1 when $\frac{f_s}{f_c} = 53.3$ *i. e.*, $\frac{16\,000}{300}$ or $\frac{20\,000}{375}$	y_1 when $\frac{f_s}{f_c} = 49.3$ *i. e.*, $\frac{16\,000}{325}$ or $\frac{20\,000}{406}$	y_1 when $\frac{f_s}{f_c} = 32$ *i. e.*, $\frac{16\,000}{500}$ or $\frac{20\,000}{625}$	y_1 when $\frac{f_s}{f_c} = 24.6$ *i. e.*, $\frac{16\,000}{650}$ or $\frac{20\,000}{810}$
37.5	0.32d	0.37d	0.41d	0.43d	0.54d	0.60d
33.3	0.29d	0.34d	0.39d	0.40d	0.51d	0.57d
30.0	0.27d	0.32d	0.36d	0.38d	0.48d	0.55d
27.3	0.26d	0.30d	0.34d	0.36d	0.46d	0.52d
25.0	0.24d	0.28d	0.32d	0.34d	0.44d	0.50d
23.1	0.22d	0.27d	0.30d	0.32d	0.42d	0.48d
21.4	0.21d	0.25d	0.29d	0.30d	0.40d	0.47d
20.0	0.20d	0.24d	0.27d	0.29d	0.38d	0.45d
18.8	0.19d	0.23d	0.26d	0.28d	0.37d	0.43d
17.6	0.18d	0.22d	0.25d	0.26d	0.36d	0.42d
16.7	0.17d	0.21d	0.24d	0.25d	0.34d	0.40d
15.8	0.17d	0.20d	0.23d	0.24d	0.33d	0.39d
15.0	0.16d	0.19d	0.22d	0.23d	0.32d	0.38d
14.3	0.15d	0.18d	0.21d	0.23d	0.31d	0.37d
13.6	0.15d	0.18d	0.20d	0.22d	0.30d	0.36d
13.0	0.14d	0.17d	0.20d	0.21d	0.29d	0.35d
12.5	0.14d	0.16d	0.19d	0.20d	0.28d	0.34d
12.0	0.13d	0.16d	0.19d	0.20d	0.27d	0.33d

In arriving at the ratio of moduli, it was assumed that the modulus of elasticity of mild steel was about 30 000 000, and that the modulus of elasticity of cinder concrete varied from about 800 000 to 2 500 000. y_1 is given for this range of moduli and for extreme fiber stresses on the concrete of 200, 250, 300, 500, and 650 lb. per sq. in. Thus, it will be found that Table 5 will cover values of y_1 for the ordinary assumptions for stone concrete, as well as for cinder concrete.

Thickness of Concrete.—Having the values of y_1, the carrying capacity of reinforced cinder concrete slabs for varying spans and varying loads can be tabulated.

Let W = total floor load, in pounds per square foot;
" L = span of slab between supports, in feet;
" M = bending moment from loading;
" R = resisting moment of slab.

Then, the bending moment (for simply supported slabs), in inch-pounds, coming on the slab is,

$$M = \frac{12\, W\, L^2}{8},$$

the resisting moment (Figs. 16 and 17) is:

$$R = M = 12 \times \frac{y_1}{2} f_c \left(\frac{2}{3} y_1 + y_2\right),$$

$$\text{hence, } \frac{W L^2}{4} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \dots\dots\dots\dots\dots (2)$$

For the outside spans of continuous slabs:

$$\frac{W\, L^2}{5} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \dots\dots\dots\dots\dots (3)$$

For slabs continuous throughout:

$$\frac{W\, L^2}{6} = y_1 f_c \left(\frac{2}{3} y_1 + y_2\right) \dots\dots\dots\dots\dots (4)$$

If $\frac{E_s}{E_c} = 30$, that is, $\frac{30\ 000\ 000}{1\ 000\ 000}$, and $\frac{f_s}{f_c} = 80$, that is, $\frac{16\,000}{200}$, then, for slabs simply supported (see values of y_1 and y_2 in Table 5):

$$\frac{W\, L^2}{8} = 24.57\ d^2, \text{ and}$$

$$d = 0.0713\ L \sqrt{W} \dots\dots\dots\dots\dots (5)$$

$$T = d + 1 \dots\dots\dots\dots\dots (6)$$

Equations 5 and 6 are for the conditions shown in Table 8.

The values for d (and for T), the thickness of concrete slabs given in other tables, were derived by substituting the proper values of y_1 and y_2, in terms of d, in Equations 2 and 4, and reducing to Equations 5 and 6.

Of course, the thicknesses of the slabs, quantities of reinforcement, and carrying capacities are based on the relative quantities of concrete and steel. An excess of concrete or an excess of steel will alter the position of the neutral axis (Equation 1) and the carrying capacity.

In these calculations the working values of f_c, etc., are about one-quarter of the ultimate; that is, at a point in the stress-strain

TABLE 6.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\ 000}{300} = 53.3$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.36\ d$; $y_2 = 0.64\ d$, FROM TABLE 5.

Span, in feet.	TOTAL UNIFORM LOADS, IN POUNDS PER SQUARE FOOT.															
	120		150		175		200		225		250		300		350	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.8	0.39	4.1	0.43	4.4	0.47	4.6	0.50	4.8	0.52	5.1	0.55	5.4	0.61	5.8	0.66
5.5	4.1	0.43	4.5	0.48	4.7	0.51	5.0	0.55	5.2	0.58	5.5	0.62	5.9	0.68	6.3	0.73
6.0	4.4	0.47	4.8	0.52	5.1	0.55	5.4	0.61	5.6	0.63	5.9	0.68	6.3	0.73	6.8	0.80
6.5	4.7	0.51	5.1	0.57	5.4	0.61	5.7	0.65	6.0	0.69	6.3	0.73	6.8	0.80	7.2	0.86
7.0	4.9	0.54	5.4	0.61	5.7	0.65	6.1	0.70	6.4	0.75	6.7	0.79	7.2	0.86	7.7	0.92
7.5	5.2	0.58	5.7	0.65	6.1	0.70	6.4	0.75	6.8	0.80	7.1	0.84	7.7	0.92	8.2	0.99
8.0	5.5	0.62	6.0	0.69	6.4	0.74	6.8	0.80	7.2	0.86	7.5	0.90	8.1	0.98	8.7	1.06

$d = 0.0513\ l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = 0.138 d.

TABLE 7.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\ 000}{250} = 64$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.32\ d$; $y_2 = 0.68\ d$, FROM TABLE 5.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.													
	120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	4.2	0.33	4.6	0.37	4.9	0.40	5.2	0.43	5.4	0.45	5.7	0.48	6.1	0.52
5.5	4.6	0.37	5.0	0.41	5.3	0.44	5.6	0.47	5.9	0.50	6.1	0.52	6.6	0.57
6.0	4.9	0.40	5.3	0.44	5.7	0.48	6.0	0.51	6.3	0.54	6.6	0.57	7.1	0.62
6.5	5.2	0.43	5.6	0.47	6.1	0.52	6.4	0.55	6.7	0.58	7.1	0.62	7.6	0.67
7.0	5.5	0.46	6.1	0.52	6.5	0.56	6.8	0.59	7.2	0.63	7.5	0.66	8.1	0.72
7.5	5.8	0.49	6.4	0.55	6.9	0.60	7.3	0.64	7.6	0.67	8.0	0.71	8.7	0.79
8.0	6.2	0.53	6.8	0.59	7.2	0.63	7.7	0.68	8.1	0.72	8.5	0.76	9.2	0.84

$d = 0.059\ l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = 0.102 d.

TABLE 8.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f}{f_c} = \frac{16\,000}{200} = 80$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.27\,d$; $y_2 = 0.73\,d$, FROM TABLE 5.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.									
	120		150		175		200		225	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	4.9	0.27	5.4	0.30	5.7	0.32	6.0	0.34	6.3	0.36
5.5	5.3	0.29	5.8	0.33	6.2	0.36	6.5	0.38	6.9	0.41
6.0	5.7	0.32	6.2	0.36	6.7	0.39	7.0	0.41	7.4	0.44
6.5	6.1	0.35	6.7	0.39	7.1	0.42	7.5	0.45	7.9	0.47
7.0	6.5	0.38	7.1	0.42	7.6	0.45	8.1	0.49	8.5	0.52
7.5	6.9	0.40	7.5	0.45	8.1	0.49	8.6	0.52	9.0	0.55
8.0	7.2	0.43	8.0	0.48	8.5	0.52	9.1	0.56	9.5	0.58

$d = 0.0713\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.068\,d$.

TABLE 9.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{300} = 53.3$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.36\,d$; $y_2 = 0.64\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.															
	120		150		175		200		225		250		300		350	
	T, Thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.3	0.32	3.6	0.36	3.8	0.39	4.0	0.41	4.2	0.44	4.3	0.46	4.6	0.50	4.9	0.54
5.5	3.5	0.35	3.8	0.39	4.0	0.41	4.3	0.46	4.5	0.48	4.6	0.50	5.0	0.55	5.3	0.59
6.0	3.8	0.39	4.1	0.43	4.3	0.45	4.6	0.50	4.8	0.52	5.0	0.55	5.4	0.61	5.7	0.65
6.5	4.0	0.41	4.3	0.46	4.6	0.50	4.9	0.54	5.1	0.57	5.3	0.59	5.7	0.65	6.1	0.70
7.0	4.2	0.44	4.6	0.50	4.9	0.54	5.2	0.58	5.4	0.61	5.6	0.64	6.1	0.70	6.5	0.76
7.5	4.4	0.47	4.9	0.54	5.2	0.58	5.5	0.62	5.7	0.65	6.0	0.69	6.5	0.76	6.9	0.81
8.0	4.7	0.51	5.1	0.57	5.4	0.61	5.8	0.66	6.0	0.69	6.3	0.73	6.8	0.80	7.3	0.87

$d = 0.042\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.138\,d$.

TABLE 10.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{250} = 64$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.32\,d$; $y_2 = 0.68\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.													
	120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.6	0.28	4.0	0.31	4.2	0.33	4.4	0.35	4.6	0.37	4.8	0.39	5.2	0.43
5.5	3.9	0.30	4.3	0.34	4.5	0.36	4.8	0.39	5.0	0.41	5.2	0.43	5.6	0.47
6.0	4.2	0.33	4.5	0.36	4.8	0.39	5.1	0.42	5.3	0.44	5.6	0.47	6.0	0.51
6.5	4.4	0.35	4.8	0.39	5.2	0.43	5.4	0.45	5.7	0.48	6.0	0.51	6.4	0.55
7.0	4.7	0.38	5.1	0.42	5.5	0.46	5.8	0.49	6.1	0.52	6.3	0.54	6.9	0.60
7.5	5.0	0.41	5.4	0.45	5.8	0.49	6.1	0.52	6.4	0.55	6.7	0.58	7.3	0.64
8.0	5.2	0.43	5.7	0.48	6.1	0.52	6.5	0.56	6.8	0.59	7.1	0.62	7.7	0.68

$d = 0.0482\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.102\,d$.

TABLE 11.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{200} = 80$; $\frac{E_s}{E_c} = 30$; $y_1 = 0.27\,d$; $y_2 = 0.73\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.													
	120		150		175		200		225		250		300	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	4.2	0.22	4.6	0.25	4.8	0.26	5.1	0.28	5.4	0.30	5.6	0.32	6.0	0.36
5.5	4.5	0.24	4.9	0.27	5.2	0.29	5.5	0.31	5.8	0.33	6.0	0.34	6.5	0.38
6.0	4.8	0.26	5.3	0.30	5.6	0.32	5.9	0.34	6.2	0.36	6.5	0.38	7.0	0.41
6.5	5.1	0.28	5.6	0.32	6.0	0.34	6.3	0.36	6.7	0.39	7.0	0.41	7.5	0.45
7.0	5.4	0.30	6.0	0.34	6.4	0.37	6.7	0.39	7.1	0.42	7.4	0.44	8.0	0.48
7.5	5.8	0.33	6.3	0.36	6.8	0.40	7.2	0.43	7.5	0.45	7.9	0.47	8.5	0.52
8.0	6.1	0.35	6.7	0.39	7.1	0.42	7.6	0.45	8.0	0.48	8.3	0.50	9.0	0.55

$d = 0.058\,l\,\sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.068\,d$.

TABLE 12.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{500} = 32$; $\frac{E_s}{E_c} = 15$; $y_1 = 0.32\ d$; $y_2 = 0.68\ d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	Total Uniform Load, in Pounds per Square Foot. 150		175		200		225		250		300		350		400	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	3.1	0.43	3.3	0.47	3.4	0.49	3.6	0.53	3.7	0.55	4.0	0.61	4.2	0.65	4.4	0.69
5.5	3.3	0.47	3.5	0.51	3.6	0.53	3.8	0.57	4.0	0.61	4.3	0.67	4.5	0.71	4.7	0.76
6.0	3.5	0.51	3.7	0.55	3.9	0.59	4.1	0.63	4.2	0.65	4.6	0.73	4.8	0.78	5.1	0.84
6.5	3.7	0.55	3.9	0.59	4.1	0.63	4.3	0.67	4.5	0.71	4.8	0.78	5.2	0.86	5.4	0.90
7.0	4.0	0.61	4.1	0.63	4.4	0.69	4.6	0.73	4.8	0.78	5.1	0.84	5.5	0.92	5.8	0.98
7.5	4.2	0.65	4.4	0.69	4.6	0.73	4.8	0.78	5.0	0.82	5.4	0.90	5.8	0.98	6.1	1.04
8.0	4.4	0.69	4.6	0.73	4.8	0.78	5.1	0.84	5.3	0.88	5.7	0.96	6.1	1.04	6.4	1.10
8.5	4.6	0.73	4.8	0.78	5.1	0.84	5.3	0.88	5.6	0.94	6.0	1.02	6.4	1.10	6.8	1.18
9.0	4.8	0.78	5.0	0.82	5.3	0.88	5.6	0.94	5.8	0.98	6.3	1.08	6.8	1.18	7.1	1.25
9.5	5.0	0.82	5.3	0.88	5.6	0.94	5.8	0.98	6.1	1.04	6.6	1.14	7.1	1.25	7.5	1.33
10.0	5.2	0.86	5.5	0.92	5.8	0.98	6.1	1.04	6.4	1.10	6.9	1.20	7.4	1.31	7.8	1.39

$d = 0.34\ l\ \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.204\ d$.

curve where the modulus of elasticity of concrete is supposed to be constant. If calculations are made for values about the elastic limit, then, according to some investigations, the moduli are variable in the concrete from the extreme fiber to the neutral axis, and the stress diagram would be somewhat like that shown in Fig. 18. With the latter assumption, the resisting moment is $12 \times f_c \times \frac{2}{3} y_1 \left(\frac{5}{8} y_1 + y_2\right)$, instead of $12 \times f_c \times \frac{y_1}{2} \left(\frac{2}{3} y_1 + y_2\right)$.

Quantity of Steel Reinforcement.—The force from the concrete (12 in. wide) resisting the bending moment due to the loading is $12 \times f_c \times \frac{y_1}{2}$; this belongs to a couple, and must equal the resistance of $f_s \times a$ in the steel. Hence, $a = 6 \frac{f_c}{f_s} y_1$, in square inches, or, in pounds per square foot,

$$a = 20.4 \frac{f_c}{f_s} y_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Substituting the values of y_1 and $\frac{f_c}{f_s}$ found in connection with each table, the quantity of reinforcement for each thickness of slab is derived.

TABLE 13.—THICKNESS OF SLAB AND QUANTITY OF STEEL REQUIRED WITH THE FOLLOWING LOADS, AND WHEN $\frac{f_s}{f_c} = \frac{16\,000}{650} = 24.6$; $\frac{E_s}{E_c} = 15$; $y_1 = 0.38\,d$; $y_2 = 0.62\,d$, FROM TABLE 5. THICKNESS INCLUDES 1 IN. FROM CENTER OF STEEL TO BOTTOM OF SLAB.

Span, in feet.	TOTAL UNIFORM LOAD, IN POUNDS PER SQUARE FOOT.															
	150		175		200		225		250		300		350		400	
	T, Total thickness of slab, in inches.	*W*, Quantity of steel, in pounds per square foot of area.	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*	*T.*	*W.*
5.0	2.7	0.54	2.9	0.60	3.0	0.63	3.1	0.66	3.2	0.69	3.4	0.76	3.6	0.82	3.8	0.88
5.5	2.9	0.60	3.0	0.63	3.2	0.69	3.3	0.72	3.4	0.76	3.7	0.85	3.9	0.91	4.1	0.98
6.0	3.1	0.66	3.2	0.69	3.4	0.76	3.5	0.79	3.7	0.85	3.9	0.91	4.1	0.98	4.4	1.07
6.5	3.2	0.69	3.4	0.76	3.6	0.82	3.7	0.85	3.9	0.91	4.2	1.01	4.4	1.07	4.6	1.13
7.0	3.4	0.76	3.6	0.82	3.8	0.88	3.9	0.91	4.1	0.98	4.4	1.07	4.7	1.17	4.9	1.23
7.5	3.6	0.82	3.8	0.88	4.0	0.94	4.1	0.98	4.3	1.04	4.6	1.13	4.9	1.23	5.2	1.32
8.0	3.7	0.85	4.0	0.95	4.2	1.01	4.3	1.04	4.5	1.10	4.9	1.23	5.2	1.32	5.5	1.42
8.5	3.9	0.91	4.1	1.00	4.4	1.07	4.6	1.13	4.8	1.20	5.1	1.29	5.4	1.39	5.8	1.51
9.0	4.1	0.98	4.3	1.04	4.6	1.13	4.8	1.20	5.0	1.26	5.4	1.39	5.7	1.48	6.0	1.57
9.5	4.3	1.04	4.5	1.10	4.8	1.20	5.0	1.26	5.2	1.32	5.6	1.45	6.0	1.57	6.3	1.67
10.0	4.4	1.07	4.7	1.17	4.9	1.23	5.2	1.32	5.4	1.39	5.8	1.51	6.2	1.64	6.6	1.76

$d = 0.028\, l \sqrt{W}$ (W = load, in pounds per square foot; l, in feet). Weight of steel, in pounds = $0.315\, d$.

Excessive Concrete or Steel.—To note the effect of excessive concrete or of excessive reinforcement on the carrying capacity, observe y_1, Equation 1. Assuming the working stress in the steel to be constant, y_1, in Table 5, increases with an increase in extreme fiber stress and decreases with an increase in the modulus of the concrete. Some designers use cinder concrete with an excess of steel in order to keep the slab as thin as possible.

Example 1.—Suppose we are limited to a working extreme fiber stress of 250 lb. per sq. in., are designing for a total floor load of 150 lb. per sq. ft. on a 6 ft. 0-in. span (simply supported slabs), but wish to limit the thickness of the slab to 4 in. (3 in. effective depth).

In this case, the extreme fiber stress being fixed at 250 lb. per sq. in., and the ratio of moduli being assumed as constant at 30, the variable to be sought in determining y_1 is the stress in the steel. Suppose we use sufficient reinforcement to bring y_1 midway between the center of the steel and the top of the slab.

Referring to Equation 1, and substituting values for d (3 in.), m (30), and y_1 ($1\frac{1}{2}$), we have $\frac{3}{2} = \frac{3}{\frac{n}{30} + 1}$; but $n = \frac{f_s}{f_c} = \frac{f_s}{250}$; hence, $f_s = 7\,500$.

Therefore, in order to bring the neutral axis midway in the effective depth of the slab and not exceed 250 lb. per sq. in. in extreme fiber stress, the reinforcement must not be strained beyond an amount given by a stress of 7 500 lb. per sq. in. The weight of reinforcement in this case will be (Equation 7): $a = 1.02$ lb. per sq. ft. Now, with steel stressed to 16 000 lb. per sq. in., as assumed in Table 7, the total thickness of the slab would have been 5.3 in., but the quantity of steel would then have been only 0.44 lb. per sq. ft. Hence, 0.58 lb. per sq. ft. is required to save 1.3 in. in thickness of cinder concrete. With reinforcing steel at $2\frac{1}{2}$ cents per lb., the 0.58 lb. of excess steel will cost 1.45 cents per sq. ft.; and the saving effected will be 1.3 in. of concrete.

Example 2.—Suppose we have a 6 ft. 0-in., simply supported slab, and are using a cinder concrete having a working capacity of 250 lb. per sq. in. of extreme fiber stress (Table 7); and, for some reason, we wish to use a certain reinforcement, a little light in area, to take the tension at 16 000 lb. per sq. in. We are not allowed to exceed the 16 000 lb., and, therefore, must thicken the slab, to more than the normal thickness given in Table 7; that is, we must use an excess of concrete.

If the total floor load is 150 lb. per sq. ft., we find in Table 7 that the slab will be 5.3 in. thick, and the reinforcement will weigh 0.44 lb. per sq. ft. of floor.

Assume that the reinforcement we wish to use weighs only 0.36 lb. per sq. ft. We see by Equation 7 that a decrease in the section of the reinforcement may be effected by a decrease in the stress on the extreme fiber of the concrete. Therefore, looking in Table 8, which has a less extreme fiber stress, we find that, with a slab 6.2 in. thick, the reinforcement is 0.36 lb. per sq. ft. of floor.

If a less quantity of reinforcement had been desired, then calculations using lower extreme fiber stresses, similar to the calculations used for making these tables, would have been necessary.

Stresses in Approved Floor Slabs.—The location of the neutral axis for any particular thickness of slab is a function of the relative stresses in the materials, and of the moduli of elasticity. Referring to Fig. 7, assume that the ratio of moduli remains constant at 30, and that, $\frac{f_s}{f_c} = 64$, that is $\frac{16\,000}{250}$ or $\frac{20\,000}{312\frac{1}{2}}$, and that the slab tested is continuous; we find in Table 10 (the table using these assumptions) that the neutral axis is $0.32d$ from the top of the slab when the concrete and steel are proportioned for these stresses. For a 4-in. slab, therefore, the neutral axis would be 0.96 in. from the top of the slab. In Table 10, a normal 4-in. slab is good for a total floor load of 150 lb. per sq. ft. on a 5 ft. 0-in. span, with a reinforcement of 0.31 lb. per sq. ft.; but in this 4-in. slab, of 8 ft. 0-in. span (Fig. 7), as approved, No. 5 wires at 3-in. centers are used. This makes the actual reinforcement about 50% greater than the normal reinforcement for Table 10.

This relative excess of steel (reducing the comparative strain coming on the reinforcement) will lower the neutral axis below that given in Table 10. The strain on the steel, $\frac{f_s}{E_s}$, therefore, is in this case, $\frac{2}{3} \times \frac{16\,000}{30\,000\,000}$ (Fig. 16). Then, the neutral axis, on account of this excess of steel, will be located $0.41d$ or 1.23 in. from the top of the slab—instead of 0.96 in. where there is no excess of steel.

According to the ruling of the New York City Building Department, the 4 in. of cinder concrete weighs 32 lb. per sq. ft. of area. Therefore, the approval of this floor construction is for a total floor load of $250 + 32 = 282$ lb. per sq. ft.

Therefore, the bending moment, from this total load (if the slab is called continuous) is $\frac{W L^2}{1}$, or, 18 048 in-lb.

This is equal to the resisting moment (one force of the couple being $12\ f_c\ \frac{1.23}{2}$) having a lever arm, $\frac{2}{3} \times 1.23 + 1.77$ in., or $19.11 \times f_c$, in inch-pounds. Therefore, $f_c = 944$ lb. per sq. in., which

is the extreme fiber stress on the concrete. The area of the reinforcement being 0.134 sq. in., $f_s = 52\,000$ lb. per sq. in. in the steel.

If, instead of considering the slab continuous, we take an end slab,* as is done with stone concrete, we have a moment of $\frac{12\ W\ L^2}{10}$, or 21 658 in-lb. In this case the extreme fiber stress will be 1 133 lb. per sq. in., and the stress on the steel, 62 000 lb. per sq. in.

If we take the condition of reinforcement simply laid over the supports, as shown in Figs. 14 and 15, then the slab is simply supported at the ends, and the load moment becomes $\frac{12\ W\ L^2}{8}$, or 27 072 in-lb. This latter condition gives an extreme fiber stress of 1 417 lb. per sq. in., and a steel stress of 78 000 lb. per sq. in.

Of course, under such abnormal stresses the modulus of elasticity for cinder concrete, probably, does not remain constant, and the resistance diagram may assume a shape similar to Fig. 18. This would slightly modify the foregoing deductions. However, even if an indefinite number of assumptions were made regarding the ratio of moduli of elasticities of wire and of cinder concrete, ratios of stresses in concrete and steel, and the variable support of slabs, etc., the foregoing deduced stresses would still be found to represent approximately correct conditions.

Without further attempt to lengthen this inquiry (by considering an indefinite number of cases and conditions), the writer would call attention to the comparison of the thickness of stone concrete slabs with approved cinder concrete slabs under similar conditions. It is found that, for the same conditions as the approved cinder concrete slab (Fig. 7) discussed herein, the stone concrete would be required to be about 1 in. thicker than the cinder concrete.

Recommendations.—

1.—The practice of testing cinder concrete, arbitrarily, under the most favorable conditions, and then recommending the results of these tests to be used under the most unfavorable conditions which may exist in a building, is dangerous and entirely unnecessary.

2.—Unit stresses for the design of reinforced cinder concrete are of more importance for its proper use than the unit stresses

* Progress Report, Special Committee on Concrete and Reinforced Concrete, *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 385.

used for reinforced stone concrete, because of the greater variableness of the former.

3.—There are, at present, sufficient precedents for choosing some unit stresses for the use of reinforced cinder concrete slabs in each locality where this material is being used.

4.—It is practical, and of little cost for each city, to establish ultimately proper working stresses for reinforced cinder concrete, based on tests with the materials used in their locality.

5.—With cinder concrete used according to proper working stresses, the construction will be removed from the realm of mystification and of misrepresentation to the field of Civil Engineering.

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CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION.

By W. C. Hammatt, M. Am. Soc. C. E.

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Summary.

This paper describes suburban and country road conditions in California and the progress in road building in that State during the past few years. The State contains plentiful supplies of good road materials, rock, gravel, asphaltic oils, etc., and there are several cement factories.

The water-bound and oiled macadam roads are described and also concrete and asphaltic pavements; and details are given as to the good and bad features of the bases and the wearing surfaces. Some costs of asphaltic pavements are included.

With exceptional advantages in regard to road materials, California is, nevertheless, in its infancy in the study of road construction. Until about six years ago, practically no permanent road surfacing had been done outside of the municipalities. In the different counties some of the more important roads had been graveled, and a great many of the county roads had been oiled, but these measures

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were taken more for the purpose of maintenance than as permanent construction work. The graveling was generally done during the rainy season in order to make the roads passable, and as the gravel was gradually worked down into the mud it was replaced by more until the depth of penetration of the surface water was reached, when the road became a good gravel highway. The oiling was generally done in dry weather, and the oil and dust formed a thin crust on the surface which was broken through by heavy traffic, and if not promptly repaired the road was soon impassable.

The southern counties of the State, which have ever been the leaders in public improvements, had expended some funds prior to the general awakening in 1908. These counties developed certain classes of cheap construction (to be described later), which, for a time, gave them an enviable reputation for roads. In 1908, however, the northern counties awakened to the need of better roads, and bond issues were passed in several of these counties for the purpose of putting permanent surfaces on their principal roads. The idea of a State highway system was developed in 1911, and, in 1912, a State Highway Commission was created and a bond issue of $18 000 000 was voted for the construction of a system of State highways.

Materials.

California is rich in many of the materials which go to make up the modern road. The Coast Range is prolific in trap rock. The Sierras are composed mainly of granite, though the foot-hills are of sandstone and shale; there are, however, in the foot-hills, a few quarries of good road rock. Gravel of a good quality for either road material or concrete, is found within easy distance of nearly every locality. There are four large companies manufacturing cement in the region of San Francisco Bay; mineral oil, with an asphaltum base, is obtained in eight different fields, distributed over about one-third of the State; and the largest refinery in the United States, producing asphaltum and asphaltic road oils, is located on San Francisco Bay.

Types of Road Construction.

Only the types of road surface used on country and suburban roads will be discussed in this paper, and each type will be taken up and described separately.

Water-Bound Macadam.—This type of pavement needs no description; it has been laid extensively in California, particularly about the centers of population. In many cases it has been laid to a finished thickness of as little as 4 in., in which case the coarsest size of rock is omitted, and the macadam is laid in two sizes only. It has been found that trap rock makes the best pavement, and that some of the softer varieties have the best cementing qualities. In fact, some of the best macadam pavements which the writer has ever seen have been laid with rock which would not pass the rattler test.

Where the sub-base has been properly prepared and drained, and the rolling has been thoroughly done, and where they have had the necessary sprinkling, macadam roads have stood up remarkably well even under automobile traffic. It is a deplorable fact, however, that the average road-builder underrates the necessity for care in the preparation of the sub-base.

Two facts are worthy of special note in the building of a macadam road: First, that a good macadam road cannot be built in rainy weather or when the sub-base is soft or wet from any other cause. To obtain proper cementation of the rock, there must be considerable resistance to the rolling. Second, the duration of the rolling cannot be specified in units of time per unit of area, as it depends on the nature of the rock and on various other conditions which make the judgment of the engineer in charge necessary.

Oiled Macadam.—This type of paving is peculiar to the localities producing an asphaltic oil. There is no fixed specification for its construction, but it is laid under many different ones, according to the whims of the engineer in charge. The name covers all classes of road surface in which oil is combined with road metal as a binder. The usual method of construction is to lay the road metal and roll the various layers in the same manner as for water-bound macadam, applying to each layer by a sprinkler a certain specified quantity of asphaltic road oil. In constructing this pavement, the main difference of opinion among road builders is in regard to the quantity of oil to be applied.

As a matter of fact, a very small proportion of the total area of oiled macadam pavement constructed has been successful. Under heavy traffic it ruts and waves very badly. It is the writer's opinion that, even under the best conditions, this pavement has little justifica-

tion. Oil has more lubricating than binding properties, and its introduction into a macadam pavement has a tendency to destroy the stability of the metal. The successful examples of pavements of this class have been either in localities where climatic conditions are such as to evaporate the more volatile parts of the oil, or where the application has been made in such a manner as to achieve the same result. Oiled macadam roads which have become so rutted as to be almost impassable, have been converted into fairly good pavements by scarifying and re-rolling, thereby aerating the oil and causing the evaporation of the lighter constituents.

Under this heading should properly come a patented pavement known as Petrolithic paving, which has been developed in the Southwest, and tried somewhat in other localities. It consists essentially of the incorporation with the natural soil of a certain definite proportion of rock and a certain quantity of oil, and the tamping of the mass into a crust, or pavement, by a roller containing spike-like projections which tamp the mixture from the bottom up. This pavement has been used with some success in localities where the soil is of a sandy rather than a clayey character, and where the summer temperature is high.

The cost of oiled macadam pavements, 6 in. in finished thickness, is about 75 cents per sq. yd., and that of Petrolithic pavements of the same depth about 60 cents.

Concrete Pavements.—Very little straight concrete pavement has been laid in California without a protective wearing surface. The writer knows of one experimental piece which has been laid with expansion joints, but it has been in use too short a time to be the source of any information at present. It was put down in two courses, the finish course being laid before the base course had set. The pavement is 6 in. thick, and cost about 90 cents per sq. yd.

In order to eke out an insufficient bond issue as far as possible, the State Highway Commission specified as the standard pavement, a 4-in. concrete base with a 1½-in. asphaltic wearing surface, and ordered that the base alone should be constructed at first, and either covered with a temporary coating of oil and screenings, or left unprotected until additional funds were available. Several hundred miles of this base have been laid, a large part of which has broken up into a series of slabs by shrinkage cracks, some of them being more

than 2 in. wide. One stretch, to the writer's knowledge, has gone to pieces by surface wear, but possibly this is due to poor concrete work which might have been prevented by better inspection. The cost of this base, inclusive of grading and what little protective coating has been used, has averaged 90 cents per sq. yd.

Asphaltic Pavements.—As might be expected from the fact that asphalt of a high grade is manufactured in California, the cities and suburban districts contain a high proportion of asphaltic pavements. The standard San Francisco pavement is 2 in. of sheet-asphalt on 6 in. of concrete. In the smaller suburban cities, this is modified to $1\frac{1}{2}$ in. of sheet-asphalt on 4 in. of concrete. In late years sheet-asphalt is being replaced by so-called asphalt macadam, under the various specifications of Warrenite, Topeka specifications, etc. The concrete base often has asphalt macadam substituted therefor, and there is now a tendency toward the macadam base for asphaltic pavements.

As opinions differ largely as to the respective merits of the various classes of bases and wearing surfaces, the writer will merely give the results of his experience and his opinions derived therefrom.

Bases.

(*a*).—The concrete base has the disadvantage of a high shrinkage in setting and a large coefficient of expansion. The former causes the base to crack into slabs in setting and the consequent destruction of its monolithic quality. The latter causes a movement after the wearing surface, and, consequently, cracks in the latter. Another fault, which is one of general practice in construction rather than in the materials of construction, is that the concrete is generally laid in such a manner that the surface is gently undulating rather than absolutely uniform. This causes the wearing surface to be of varying thickness and subject to unequal movement, as will be described later under that heading. This can be obviated, however, by trimming the concrete base with a template while laying.

This base is the one most commonly used, due to a popular prejudice in its favor. Most laymen and many engineers cannot get away from the idea that a rapid chemical reaction will produce a substance more durable and serviceable than rock combined according to the laws of gravity and stability.

(*b*).—The asphaltic macadam base has not given good results, for the following reasons: It has no stability, but is subject to the same faults as the too-thick asphaltic wearing surface which will be described later.

In addition, asphalt in contact with soil has a tendency to promote vegetable growth, and many pavements of this class have been heaved up and broken from root growth, where, prior to the placing of the pavement, such growth did not exist.

(*c*).—A water-bound macadam base is the best for supporting an asphaltic wearing surface, for various reasons. It has a stability due to the seating of its component parts, which is not dependent on any chemical action. It has no movement under temperature changes. It is not rigid, and, therefore, does not give the destructive reaction to impact on the surface of the asphaltic pavement, which is a feature of the concrete base. The surface is free from depressions which vary the thickness of the asphaltic wearing surface.

Wearing Surface.

(*a*).—The limits of efficacious thickness of the wearing surface are quite small. The wearing surface must be thick enough to be held in position by its own inertia, as it has but small cohesion to the base; and it must be so thin that the compression of rolling shall be communicated throughout its entire thickness. It is inadvisable to make the wearing surface less than 1½ in., or more than 2 in., in thickness. Where the asphaltic wearing surface is laid to such a thickness that the compression is not communicated throughout the entire depth, the compression is not uniform, and the subsequent adjustment, which always takes place in an asphaltic mixture of insufficient density, causes inequalities in the surface which are starting points for failure by waving.

(*b*).—The quality of the pavement and the roughness of the surface are not dependent to any great extent on the size of the coarse aggregate. The best and most durable mixtures are those containing the highest proportion of fine aggregate of 80 to 200 mesh. If the mixture is laid hot enough, and the compression is properly done, the fine material, or "mortar", will come to the surface and thoroughly fill all the voids in the coarse aggregate, which then merely serves to decrease the quantity of asphalt necessary. The only way of obtaining

the rough surface so much desired by some, is either to have insufficient fine material to fill the voids, or to lay the mixture too cold or with insufficient compression. In each case, the wearing surface will be open and spongy. Also, if the coarse aggregate projects above the general surface of the pavement, the stones are crushed or rolled by steel tires passing over them, leaving voids where disintegration may start.

The costs of the various types of asphaltic pavement are as follows:

2-in sheet-asphalt, on a 4-in. concrete base........$1.50 per sq. yd.
$1\frac{1}{2}$-in. Topeka specification, asphaltic wearing surface on a 4-in. concrete base..............$1.15 per sq. yd.
2-in. Topeka specification, asphaltic wearing surface on a 4-in. macadam base...............$1.10 per sq. yd.

The laying of an asphaltic wearing surface on an old macadam pavement, with no other preparation than the cleaning off of the fine material, has been done with great success. The cost of a $1\frac{1}{2}$-in. Topeka specification wearing surface laid on old macadam is about 50 cents per sq. yd. All the costs given are based on labor and material costs prevailing in the bay counties about San Francisco.

Another matter worthy of note is the tendency of municipal governments in California to require street improvement work in the outlying districts to follow the same lines as in the thickly settled parts. Where the property is in large tracts, the villas have their own driveways, so that there is no occasion for teams or vehicles to stop in front of the property, nevertheless most of the municipalities require the placing of cement curbs and sidewalks, and the paving of the roadway from curb to curb, as in business blocks, thus spoiling the possibilities of architectural landscape work.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

HUACAL DAM, SONORA, MEXICO.

By H. Hawgood, M. Am. Soc. C. E.

To be Presented May 20th, 1914.

Summary.

This paper describes the methods used in the design and construction of a concrete arch dam dependent on arch action for its stability. The dimensions of the dam are: crest length of curved structure, 140 ft.; radius, 76 ft.; maximum height, 100 ft.

The futility of attempting a determination of stresses by pure mathematical processes is discussed. It is pointed out that the lack of definite knowledge as to the behavior, under stress, of the rock in which the ends and base of the dam are intrenched, and to which it is joined, inhibits the application of pure mathematics, and the use of the simple formula for a thin cylinder is sustained.

For general information, and for comparative purposes, Table 1 gives the characteristic dimensions and stresses of numerous pure arch dams in various parts of the world.

The cost of the work is given, and there is also a description of the pipe line, conveying the impounded water from the dam to the place of use, together with its cost.

The works were executed in a difficult country, with impossible wagon roads, where the cement, sand, and some other materials had to be packed on animals, and where labor was disturbed by a state of war.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The concrete dam described in this paper is an arched structure dependent on arch action for its stability. The use of the arch for dams and dependence on it for stability, is old; how old is not susceptible of proof, but certainly more than a century. Some time prior to 1800 was built the Meer Allum Dam, at Hyderabad, India, a multiple-arch structure composed of a succession of brick arches, varying from 40 to 82 ft. in radius, with an aggregate length of about ½ mile and a water depth of about 40 ft. A more recent Indian example is a dam 60 ft. high, also of the multiple-arch type, at Alwar, Rajputana. In Europe the Zola Dam, near Aix, France, of a radius of 158 ft. and a height of about 120 ft., dates from 1843.

In more recent years many arch dams have been built, notably in New South Wales and South Australia. Their characteristic dimensions are given on Table 1.

In the United States, the Bear Valley Dam, Southern California, built in 1886, has in the past attracted much attention by the boldness of its design, and yet its audacity is equalled by the less known Upper Otay Dam of the San Diego Water Supply, built in 1900. The granite masonry of the Bear Valley Dam was subjected to compressive stresses of about 61 tons per sq. ft., and the concrete of the Upper Otay Dam is subjected to stresses of similar intensity. The Otay Dam is an instructive illustration of the stresses which can be sustained successfully by a horizontal arch of Portland cement concrete, mixed 1 : 2.1 : 3.4, when founded and abutting on strong, hard rock. The advisability, however, of imposing such high stresses may well be questioned.

It is of interest to note that the old Bear Valley Dam, after doing duty for 27 years, has at last survived its usefulness. It has been superseded, and at high-water stages is submerged by the water impounded by a higher dam of the inclined multiple-arch type, designed and built by J. S. Eastwood,* M. Am. Soc. C. E.

The writer has not found any record of the failure of a true arch dam; if any have failed, it would be well to have all the facts brought to light.

It will be noted that the stresses sustained by the various structures have an extremely wide range, varying from a minimum of 10.6 tons to a maximum of 61 tons per sq. ft., or from 147 lb. to 870 lb. per sq. in.

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* *Engineering News*, December 25th, 1913, Vol. 70, p. 1284.

After study of the available local materials and labor conditions, a stress of 16.3 tons per sq. ft., 226 lb. per sq. in., was adopted for the Huacal Dam. If labor experienced in concrete work had been available at Nacozari, a higher stress might have been considered. The most vigorous supervision cannot entirely eliminate the errors of uninformed labor; foremen, however competent, cannot be everywhere at once, and it is prudent to make allowance for probable inferior quality of the labor, an important factor in the proper placing of concrete.

The physical characteristics of the coarse aggregate enter largely into the question of allowable compressive stresses.

General.

The Huacal Dam was constructed during 1911-12 for the Moctezuma Copper Company, Nacozari, Sonora, Mexico (Phelps, Dodge and Company, New York), from the designs and under the general direction of the writer.

The purpose of the dam is to impound storm waters and create a gravity supply for the milling operations of the Copper Company. The water supply prior to the construction of the dam was obtained by pumping from the gravel beds of the Rio Nacozari. For a period of from 2 to 4 months of each year the waters of these gravels became so far depleted as to impose limitations on the output of the mill. The normal capacity of the mill is 400 tons of concentrates per day, and the required daily supply of water is about 1 250 000 gal. The pumping plant, now out of service, but held as an emergency reserve, consists of two 10 by 12-in. Aldrich triplex plunger pumps, electrically driven through gearing, discharging against a head of 225 ft. In round figures, the yearly cost of operating these pumps was $12 500. Decrease in cost and increase in water supply constituted the economic incentive for the dam.

Nacozari, the present southern terminus of the Ferro Carril de Nacozari, is 76.6 miles south of the International Boundary. The northern termini of the railroad are Douglas, Ariz., on the American side of the line, and Agua Prieta, on the Mexican side.

The dam is built across the canyon of Huacal Creek, about 3 miles northeast from Nacozari, at an elevation of 550 ft. above the town and 4 100 ft. above sea level. Nacozari formed the base of construction

TABLE 1.—CHARACTERISTICS

	Date of construction.	Location or name.	*R*, Radius, in feet.	*L*, Top length of segment, in feet.	Arc, in degrees.
		UNITED STATES OF AMERICA.			
a	1886........	Bear Valley, Cal......................	335	300	51° 20′
b *c*	1900........	Upper Otay, Cal......................	359	350	56° 00′
d	1907........	Crowley Creek, Ore..................	70	150	55° 50′
e	1913......	"Goodwin Dam," Stanislaus River, Cal. Twin arches, each............	135	233	100° 00′
		MEXICO.			
	1912........	Huacal, Sonora	76	140	105° 40′
		SOUTH AUSTRALIA.			
	1904........	Barrosa..............................	200	472.5	135° 20′
		NEW SOUTH WALES.			
	1897........	Parkes...............................	300	540	100° 00′
	1898........	Costamundra..........................	250	475	109° 30′
	1898........	Tamworth	250	440	100° 00′
	1899........	Wellington...........................	150	350	133° 00′
f	1899........	Mudgee...............................	253	498	112° 00′
	1899........	Wollongong	200	344	98° 30′
	1896........	Lithgow No. 1........................	100	162	93° 00′
	1897........	Picton...............................	120	112	53° 00′
	1898........	Queen Charlotte Vale.................	90	113	72° 00′
	1858 1898	Paramatta............................	160	225	80° 00′
	1906........	Lithgow No. 2........................	100	221	127° 00′
	1906........	Medlow...............................	60	81	78° 00′
	1907........	Barren Jack..........................	80	....	....
		INDIA.			
g	Prior to 1800......	Meer Allum...........................	82	178	175° 00′
		FRANCE.			
h	1843........	Zola, Aix............................	158	205	74° 20′

AUTHORITY.

a. Bear Valley Company's Original Records, also Schuyler's "Reservoirs for Irrigation, Waterpower and Domestic Supply," pp. 246-256.
b. *Engineering News*, Vol. 46, p. 125.
c. *Engineering News*, Vol. 51, p. 337.
d. *Engineering News*, Vol. 66, p. 220.
e. *Engineering News*, Vol. 70, p. 748.

supplies. Bulky pieces were transported by wagon, over a rough mountain road some 6 miles in length, with steep gradients, the maximum load being 1 ton to four good animals. Everything else, including all cement and sand, was packed on mules and burros, with loads of 300 lb. or more to the mule, and 175 lb. to the burro. The

OF CONSTRUCTED "ARCH-DAMS".

D, Depth of water, in feet.	*T*, Thickness at *D*, in feet.	*S*, Stress, in tons per square foot.	Nature of aggregate.	Class of masonry.
48.0	8.42	61.0	Granitic.	Granite masonry.
75.0	14.00	61.0	Granitic.	Concrete.
50.0	12.00	46.7		
63.0	5.16	26.6	Lava.	Concrete.
70.0	at *D*= 61.0 ft. 16.0 "	21.4	Gravel.	Concrete.
88.5	12.83	16.3	Andesitic.	Concrete.
94.0	34.00	17.3	Gneiss.	Concrete.
37.0	13.50	29.1	Granitic.	Concrete.
45.5	12.88	28.2	Granitic.	Concrete.
62.0	21.50	23.3	Granitic.	Concrete.
48.5	10.00	23.7	Conglomerate.	Concrete.
50.0	18.00	22.4	Slate.	Concrete.
41.5	11.62	22.9	Basalt.	Concrete.
38.0	11.38	11.4	Sandstone.	Concrete.
40.0	13.62	13.8	Sandstone.	Conerete.
34.0	8.65	11.7	Quartzite.	Concrete.
46.0	15.00	15.3	Sandstone.	Concrete.
78.5	24.00	10.6	Sandstone.	Concrete.
66.0	8.96	14.4	Sandstone.	Concrete.
38.0	5.00	19.0	....	Concrete.
39.0	8.50	11.8	....	Brick masonry.
119.7	41.80	14.2	....	Rubble masonry.

f. *Minutes of Proceedings*, Inst. C. E., Vol. 178.
g. *Minutes of Proceedings*, Inst. C. E., Vol. 172, p. 214.
h. Schuyler's "Reservoirs for Irrigation, Waterpower and Domestic Supply," p. 362.
The stresses, *S*, in Table 1, are computed by the cylinder formula:

$$\text{Stress, in tons (2 000 lb.) per sq. ft.} = \frac{\text{Radius (feet)} \times \text{Depth (feet)}}{32 \times \text{Thickness (feet)}}.$$

average cost of packing was about \$1.02 per 2 000 lb. The cement was packed from the railroad depot, a distance of about 6 miles, and the sand from the Nacozari River, about 4 miles. The cost per ton-mile, one way, averaged about 21¾ cents.

All prices and costs are given in United States gold values.

Catchment Area.

The catchment tributary to the dam (Fig. 3) has an area of 13 sq. miles, is roughly rectangular in form, with an extreme length of 5.7 miles and a width of 3.4 miles. It is a mountainous area, broken by numerous watercourses with deeply incised channels. The slopes leading down to the channels are steep, vegetation is sparse, the soil shallow, with much bare rock, and the storm discharges are of a torrential character.

Rainfall.

The rains commence in June. May is a dry month, and the rain records are in the best form for comparative purposes when arranged by seasons running from June 1st to May 31st of the succeeding year. A rain gauge has been kept at Nacozari since May, 1910, but no definite information as to previous rainfall can be obtained.

TABLE 2.—Rainfall at Nacozari by Seasons.

Month.	1910–11. Inches	1911–12. Inches.	1912–13. Inches.	1913–14. Inches.
June	0.51	2.66	1.17	0.08
July	4.39	3.73	4.66	4.13
August	2.32	2.25	3.59	2.77
September	0.89	4.79	0.00	
October	0.14	3.36	0.54	
November	0.33	1.38	0.83	
December	0.00	1.13	0.77	Highest rate of precipitation 3.05 in. in 3 hours, October 4th, 1911.
January	1.73	0.00	0.84	
February	1.43	0.31	4.52	
March	0.01	1.96	0.57	
April	0.60	0.20	0.53	
May	0.00	0.00	0.02	
Totals	12.35	21.77	18.04	....

No gauges are maintained other than the one at Nacozari. The rainfall over the catchment area above the dam is unquestionably greater than at Nacozari, but to what extent is as yet purely a matter of estimate and conjecture.

Run-Off.

On the basis of the probable rainfall and the topographical features of the catchment area, it was estimated that the yield of a rain season such as 1910-11 would be approximately 3 000 acre-ft. (2 700 000 gal. per day). The season of 1910-11 was classed as "dry" by those familiar with the country. This opinion is supported by the rain records of

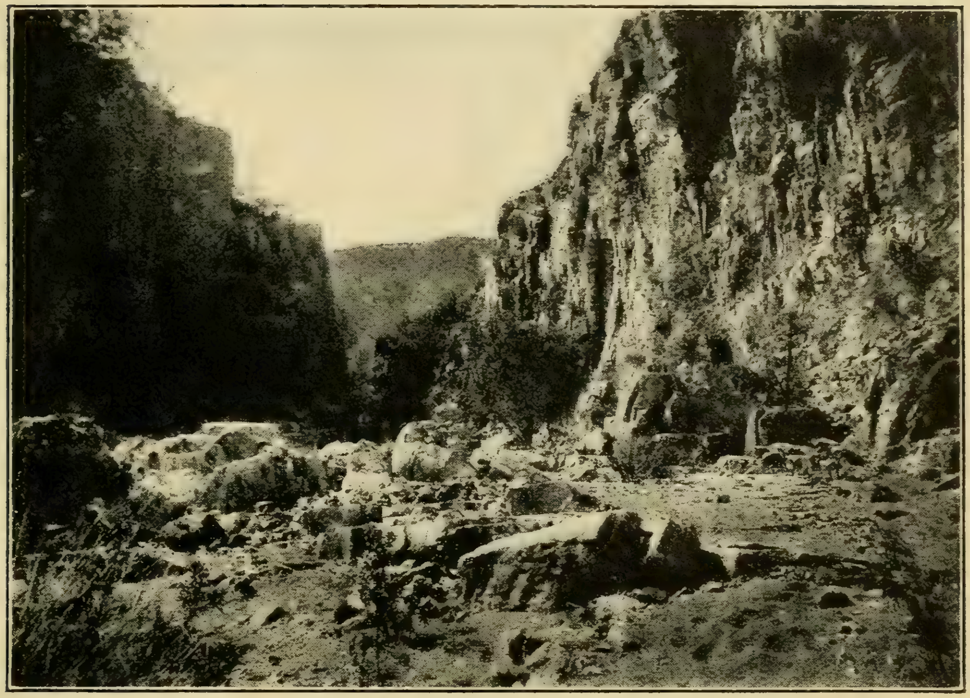


FIG. 1.—DAM SITE, LOOKING DOWN STREAM.

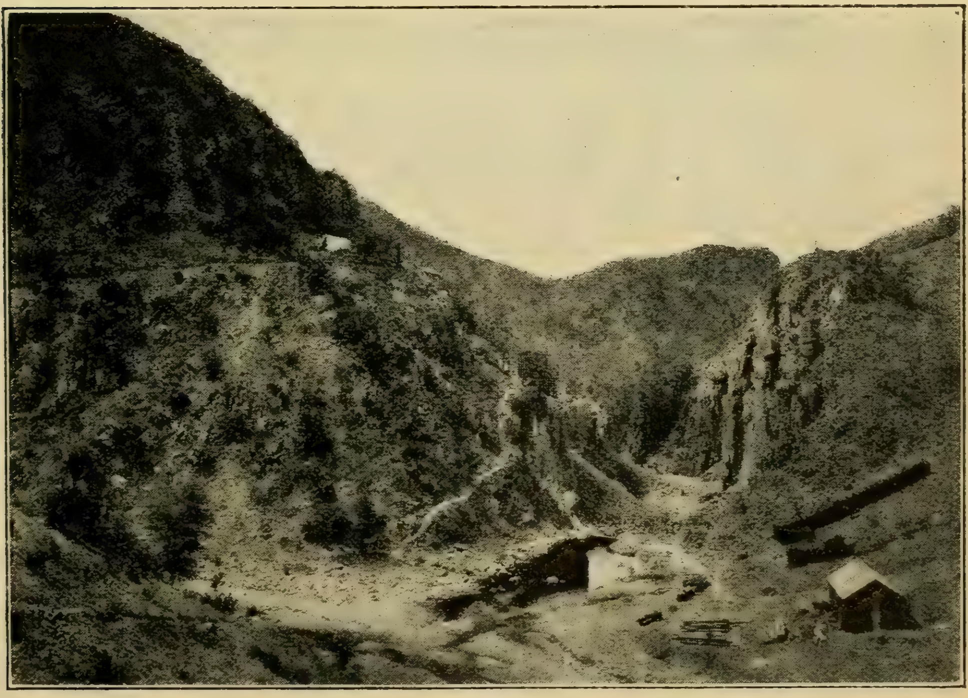


FIG. 2.—COMMENCEMENT OF WORK. LOOKING DOWN STREAM.

1911-12, 1912-13, as far as they go, but they are of too short duration to form any conclusions, and it cannot be assumed that there will not be drier seasons than 1910-11. The margin between the con-

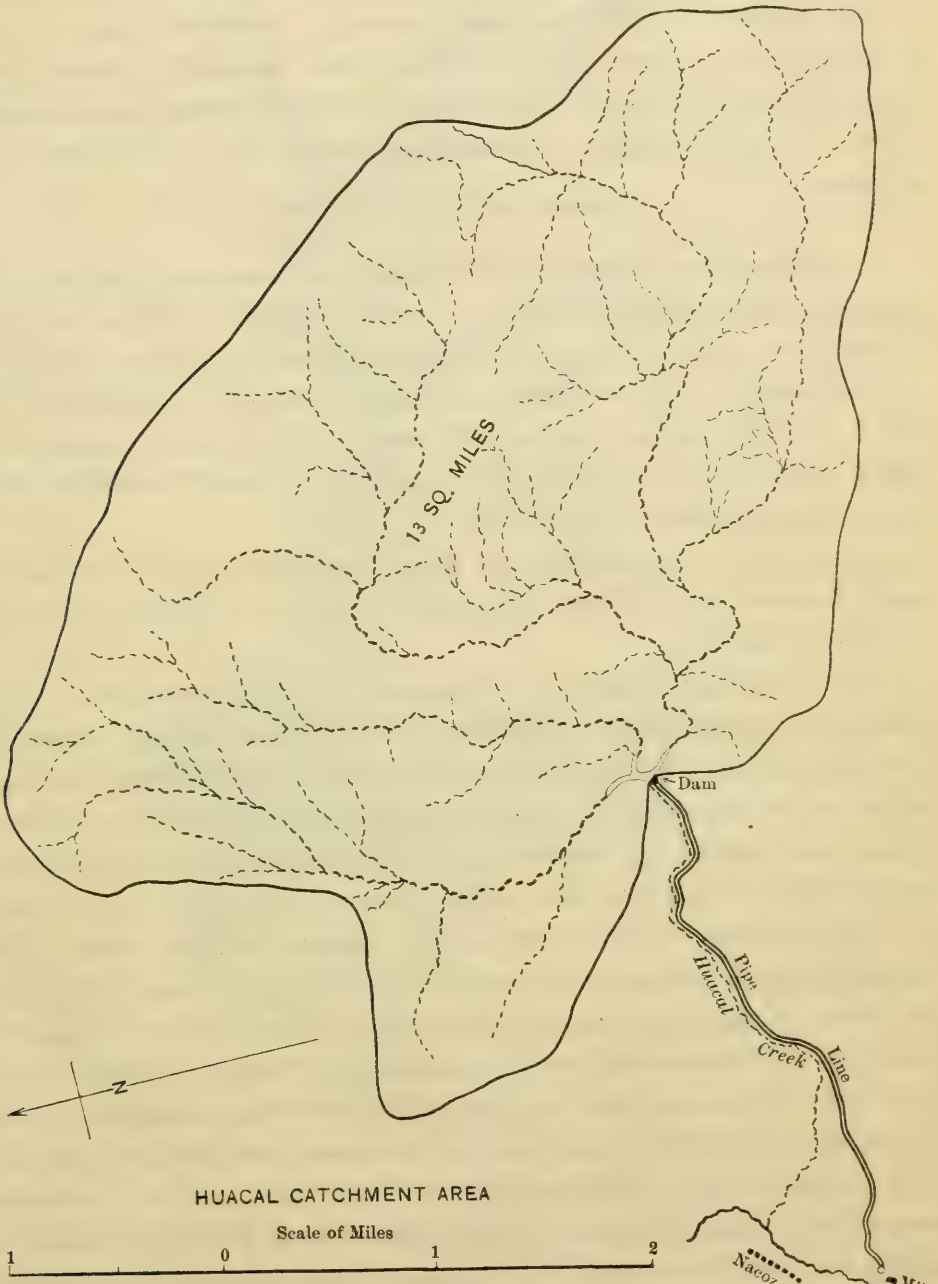


FIG. 3.

templated use of 1 400 acre-ft. annually, or 1 250 000 gal. daily, and the 3 000 acre-ft., or 2 700 000 gal. daily run-off estimated for 1910-11 is considered ample to take care of drier seasons.

On August 14th, 1910, a storm run-off was observed lasting about 1 hour, the total discharge from which was estimated at 300 acre-ft., with an average run-off rate of 255 sec-ft. per sq. mile.

On October 5th, 1911, a flood, having a maximum rate of 2 000 cu. ft. per sec., passed, without damage, over the partly finished dam, then 40 ft. high. The total discharge during this storm was estimated to be 1 240 acre-ft., with a maximum run-off rate of 154 sec-ft. per sq. mile.

Reservoir and Dam Site.

In outline, the reservoir is T-shaped, as shown by Fig. 4. The dam and outlet of the reservoir are on the stem of the T, to each side of which the water surface at high water extends for about 1 mile.

The capacity of the reservoir at the crest level of the dam is 2 700 acre-ft., with a surface area of 93 acres. It is designed, should it be found desirable to impound more water, to install tumbling flash-boards along the crest of the dam.

At the point selected for the dam the canyon had in its natural state a bottom width of from 20 to 30 ft., and, at 90 ft. above the bottom, a width of 150 ft.

The geological formation is Andesitic. In the main, the rock is hard and close-grained. The dip of the formation approaches the vertical, and the strike is across the canyon approximately at right angles to its axis, both of which are conditions favorable to minimum leakage and stability of foundation. A seam discharging a little water was disclosed in driving the outlet tunnel and in the shaft beneath the outlet tower. During the active progress of the work it was impracticable to isolate and measure this water. In April, 1912, after the water in the reservoir had arisen against the dam to a height of about 40 ft., this spring was measured and found to yield 1.5 gal. per min. The entire flow over the rock bed of the stream at a point about 100 ft. below the dam was, on the same date, 3.58 gal. per min. This would ascribe a seepage through the dam and its foundation contact of 2.1 gal. per min. In February, 1913, 10 months later, the entire flow over the stream bed was again measured and found to have decreased to 2.83 gal. per min., the depth of water against the dam having in the interim increased from 40 to 60 ft.

The seepage through the dam and its foundation contact was not precisely segregated, but was judged not to exceed 1.0 or 1.5 gal. per

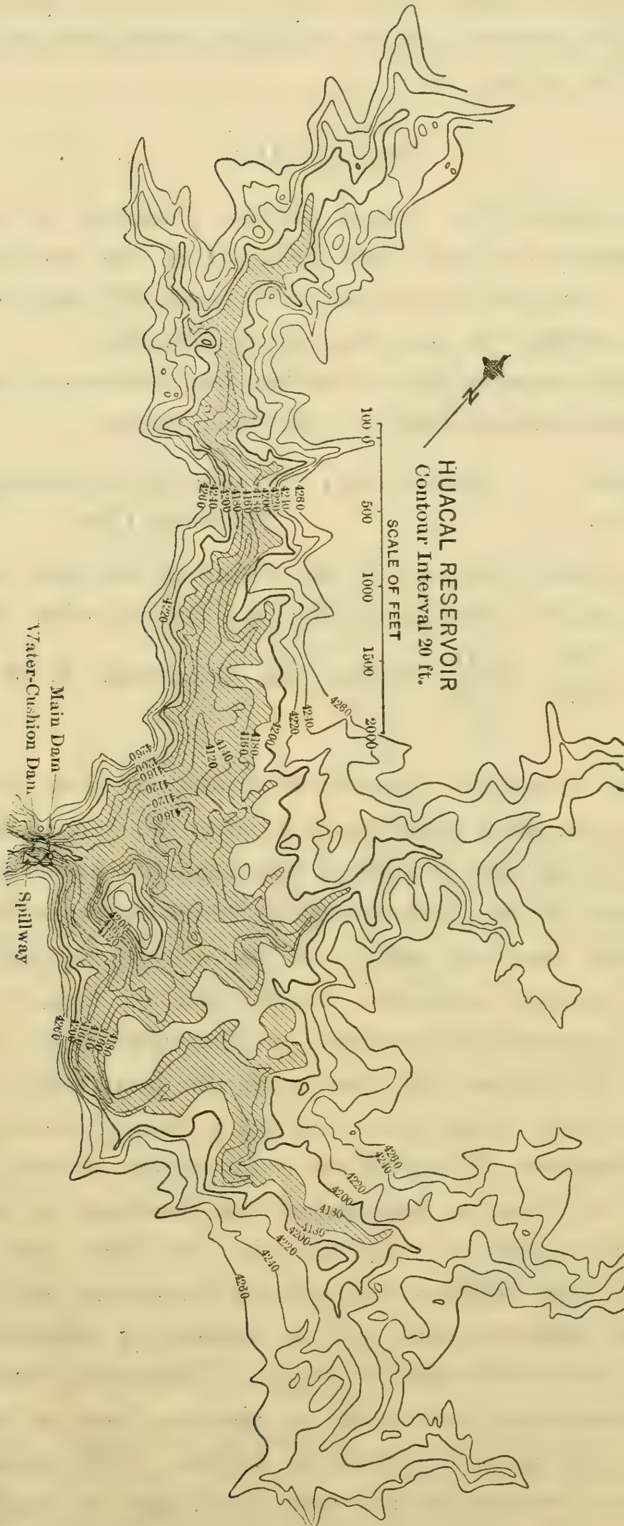

HUACAL RESERVOIR
Contour Interval 20 ft.
SCALE OF FEET
Main Dam
Water-Cushion Dam
Spillway

FIG. 4.

min. This trivial seepage speaks well for the care taken in making and placing the concrete, and in obtaining sound contact between the concrete and the native rock.

Design of Dam.

The narrowness of the canyon and the character of its rock sides and floor suggested an arch dam (Fig. 5) as the economic type for the location. Comparative studies with a rock-fill dam as a practical alternative confirmed the selection of the arch type.

The requisite cross-section of the dam was determined by the simple formula for cylinders subjected to external pressure:

$$\left.\begin{array}{l}\text{Stress, in tons}\\ \text{(2 000 lb.) per sq. ft.}\end{array}\right\} = \frac{\text{Radius (feet)} \times \text{Pressure (pounds per square foot)}}{\text{Thickness (feet)}}.$$

The values finally adopted in the design of the dam were: Stress, 16.3 tons per sq. ft.; radius, 76 ft.; and the equation for thickness became $T = \frac{76}{16.3} \times \frac{D}{32}$, whence the thickness, T, at the depth, $D = 0.146\ D$.

With these values the theoretical triangular dam section has a vertical front face, and a back face on a batter of 1¾ in. to the foot. The lower portion of the dam is built of this form, the upper portion being widened to meet the practical necessities which govern a dam top.

The formula does not satisfy all the conditions, mathematically, for the dam is neither a complete cylinder with free ends, nor is it everywhere free to expand and contract; on the contrary, it is a segment connected at its bottom and sides with the immovable, but not the less elastic, country rock; and it is not a thin cylinder, but one of appreciable thickness. The problem is impossible of exact mathematical solution, for, however approached, its treatment is based on assumptions. The break of continuity in both form and material at the line of contact between the artificial structure and the natural rock, and the unknown value of the element of deformation in the natural rock, create uncertainties which, in themselves, render a mathematical demonstration based solely on absolute facts an impossibility, for the facts are not and cannot be known. Such elements of uncertainty are not confined to arch dams alone, they are equally prevalent with gravity dams. The researches and experiments of Pearson, Otley,

Brightmore, Baker, and others have shown the accepted middle-third method of gravity section design to fall short of being absolutely correct. Nevertheless, the fact remains that dams designed on that theory,

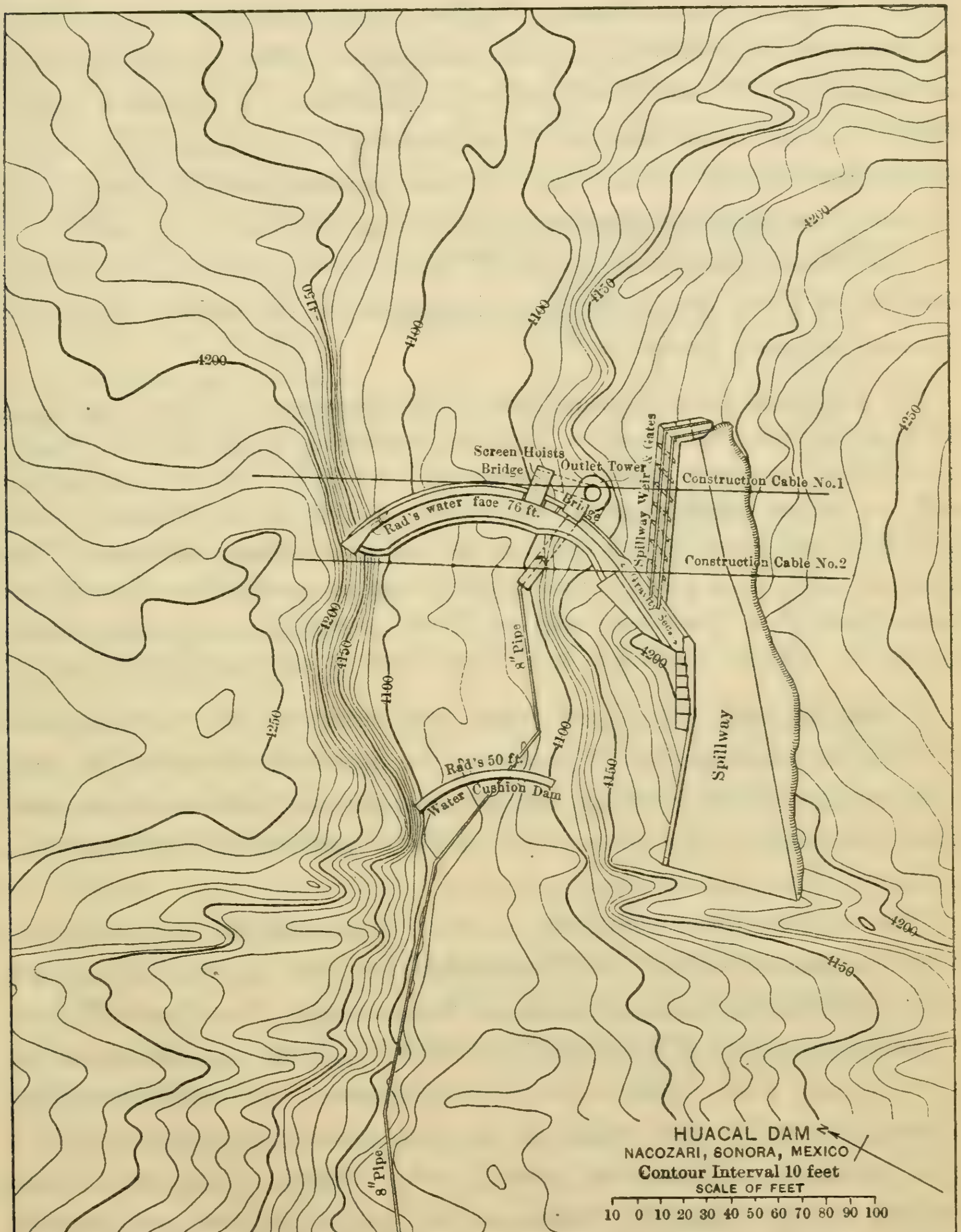


FIG. 5.

within certain recognized limitations as to intensity of stress, have been and continue to be successful. Similarly, the many Australian arch dams designed on the simple cylinder theory have been and continue to be successful.

Whatever the unknown and undeterminable stresses may be, particularly those around the foundations, the adopted formulas, empirical though they may be, have given and do give safe results, and, in the present status of knowledge as to the actual stresses in dams, their use must be considered to be more conservative than that of formulas mathematically correct but based on assumptions of unknown conditions. The successful practice of engineering is one of applied science, rather than of pure science, and the pure mathematical formula, without some modifying coefficient derived from practice, is a rarity.

The views expressed by Dr. W. C. Unwin, Past-President of the Institution of Civil Engineers, in a discussion on the subject of stresses in dams, are very pertinent:*

"It had now been shown that it was hopeless to attack the dam problem by pure mathematics. He did not think there was any theory on which an engineer relied in designing a bridge, a roof, or a retaining wall, to which mathematical objections could not be raised of the kind that had been raised against the ordinary theory of dams. The state of stress in a plate round a rivet was as complex as that in the foundation of a dam, and was as little susceptible of mathematical treatment; and yet engineers were not afraid of making riveted joints."

It may be said, with equal truth, that engineers will not be afraid to continue building gravity dams on the middle-third theory, or arch dams on the cylinder theory, and no disaster will follow, always provided that there is due recognition of the conditions of the foundation and abutments which Nature provides.

The futility of attempting a pure mathematical determination of the stresses in a dam is well exemplified by the measurements of the actual deflections of an arch dam at Barren Jack, New South Wales.†

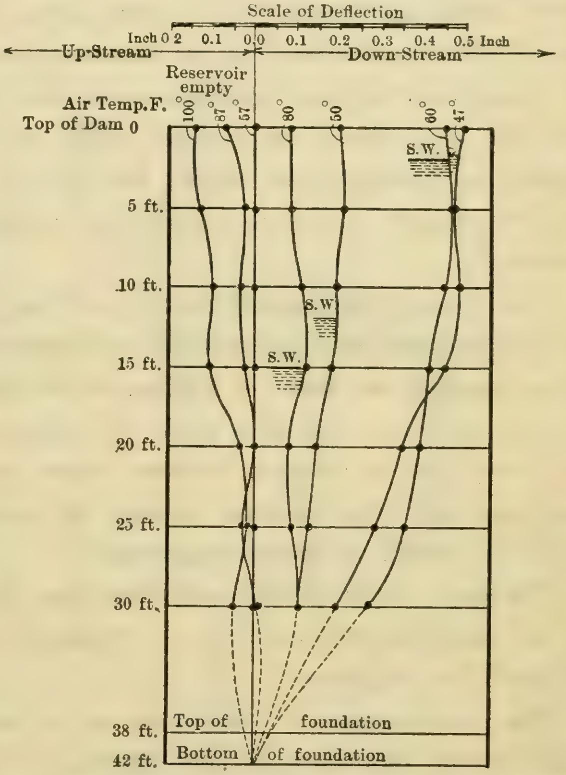
The dimensions of this dam are given in Table 1. The structure is of concrete with a light vertical reinforcement (against temperature stresses) of 20-lb. steel rails, 10 ft. from center to center and 1 ft. from the face. As the steel area is only 0.044% of the area of the concrete at the base, and 0.139% at the top of the dam, its presence is negligible, as far as bending stresses are concerned.

The deflections of the Barren Jack Dam, which are shown in the diagram, Fig. 6, were taken under varying conditions of tempera-

* *Minutes of Proceedings*, Inst. C. E., Vol. 172, p. 156.

† *Minutes of Proceedings*, Inst. C. E., Vol. 178, p. 61.

ture and water depth. The observations made when the reservoir was empty and the air temperature 57°, which approximates the closest of the observations to mean temperature, are drawn as a straight line, and the other observations are plotted in regard thereto. The curves drawn through the various observed points are brought for convenience to one common point of origin on the foundation base. This is not strictly correct, for it makes the impossible assumption that the base is absolutely rigid, and that the rock on which the dam stands is incapable of elastic deformation under the stresses transferred to it by the dam and by the weight of water supported by the rock bed immediately above the dam. Deformation of the rock undoubtedly takes place, and, such being the case, the arch action takes place in some unknown degree from the very base of the dam. That deformation of the bed-rock does take place to considerable depths was the opinion expressed by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in discussing the Coolgardie Water Supply.*



DEFLECTIONS OF BARREN JACK ARCH DAM
At different temperatures and heights of water.
Measured at 5-ft. vertical intervals from top of dam to 30-ft. below
Minutes of Proceedings, Inst. C. E. Vol. 178

FIG. 6.

Fig. 6 indicates that the bending stresses at the base are probably little if any greater than at higher points of the dam. Were the bending stresses at the bases of arch dams as great as some theories would lead us to think, many if not all the arch dams cited would have been fractured, and in particular the very thin dams of Bear Valley and the Upper Otay. That they have not been fractured is conclusive that the theories need amending.

* *Minutes of Proceedings*, Inst. C. E., Vol. 162, p. 123.

The extreme height of the Huacal Dam from base to crest is 100.25 ft. The foundation trench was excavated until sound rock was reached at depths from 5 to 15 ft. That there might be no powder cracks, hand tools only were used in the bottom of the trench. Similar precaution was exercised in the abutment trenches at each end of the dam, which were carried horizontally into the rock formation to a maximum distance of 20 ft. to insure contact with sound rock.

The front face of the dam is vertical, the back face has a variable batter changing from $1\frac{3}{4}$ in. to the foot at the bottom, to $\frac{1}{2}$ in. to the foot at the top, as shown on the cross-section, Fig. 7. As actually built, the back face does not conform with this design precisely. By a mishap, the dam about its middle height is somewhat thicker than designed, as shown by the dotted line on the cross-section. The profile was returned to the true lines with an easy batter, as shown in the drawing.

The top of the arched portion of the dam is made hollow, both to save concrete and to provide free intercommunication between the space beneath the sheet of falling water when the dam is overflowed, and the open air, and thus avoid the tremors incidental to the making and breaking of contact between water and dam with the forming and breaking of a vacuum behind the nappe when the free passage of air is wholly or partly suppressed.

The dam is not of the arch form for its entire length. For 42 ft. at its southeasterly end it is of gravity section, and in direction tangential to the curve.

The continuance of the dam on a curve throughout its entire length would have brought the direction of its easterly end too closely parallel to the axis of the canyon to have been economical or safe.

In the canyon below the main structure there is a supplementary dam designed to create at times of overflow a water cushion with a maximum depth of 20 ft. over the base of the main dam.

Spillway.

No stream discharge records being available, the determination of spillway capacity necessary for safety became largely speculative and matter of opinion. The subject was studied in conjunction with D. C. Henny, M. Am. Soc. C. E., and after weighing all available data it was concluded that provision to pass a flood of 6 000 sec-ft. would

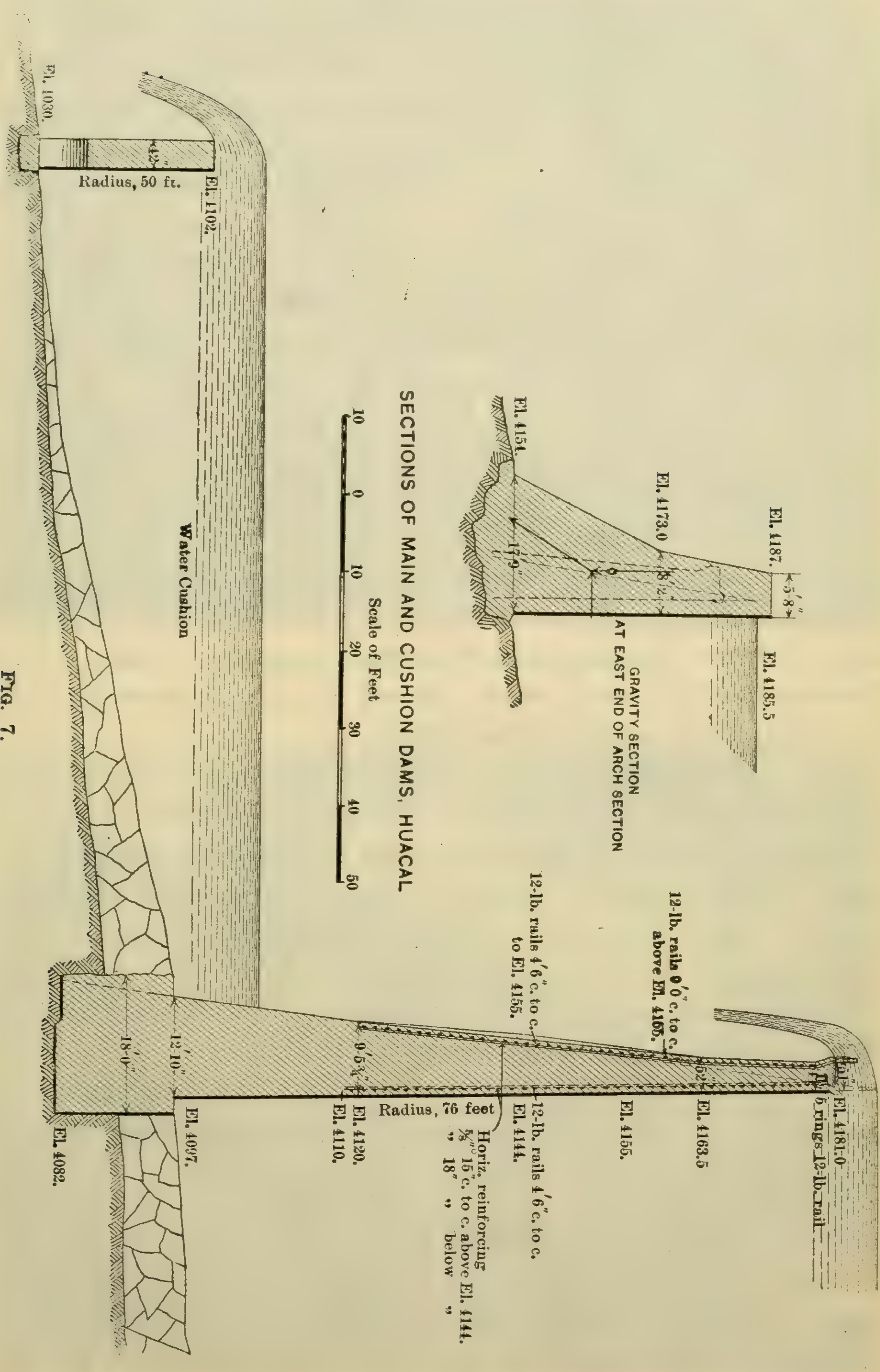

SECTIONS OF MAIN AND CUSHION DAMS, HUACAL
Scale of Feet
10 0 10 20 30 40 50
GRAVITY SECTION
AT EAST END OF ARCH SECTION
El. 4187.
5'-8"
El. 4185.5
El. 4173.0
El. 4154.
17'-0"
12-lb. rails 9'0" c. to c.
above El. 4166.
12-lb. rails 4'6" c. to c.
to El. 4155.
El. 4185.5
El. 4181.0
6 rings 12-lb. rail
El. 4163.5
El. 4155.
12-lb. rails 4'6" c. to c.
El. 4144.
Horiz. reinforcing
5/8" 15" c. to c. above El. 4144.
" 18" " below "
Radius, 76 feet
9'-6"
El. 4120.
El. 4110.
12'-10"
18'-0"
El. 4097.
El. 4082.
Water Cushion
El. 4102.
Radius, 50 ft.
El. 4020.

FIG. 7.

be ample. If this quantity is ever attained, 2 600 sec-ft. would flow over the top of the dam, and 3 400 sec-ft. through the spillway. Provision for a run-off of 6 000 sec-ft. from a catchment area of 13 sq. miles, or 452 sec-ft. per sq. mile, may be excessive, and there is some local evidence that such is the case; on the other hand, flood peaks of greater volume are not unknown in the semi-arid districts. The writer had occasion to make an exhaustive investigation of the Chase Creek, Arizona, flood of December, 1906. Chase Creek, a tributary of the Gila River, is about 200 miles north of Nacozari. Both localities have somewhat similar climates. The peak of the Chase Creek flood had a discharge rate of 647 sec-ft. per sq. mile from an area of 20 miles. The duration of the flood wave was about 40 min., with a mean rate of flow during that period of about 420 sec-ft. per sq. mile. The slopes of the country draining into Chase Creek are precipitous, averaging 1 100 ft. fall to the mile. The drainage slopes of the Huacal Basin are much flatter, and the channels less direct, consequently, the maximum discharge rate for the same rainfall will be less from the Huacal water-shed than from one of such abrupt topography as Chase Creek. Provision for 452 sec-ft. on the Huacal is probably more than the equivalent of 647 sec-ft. on Chase Creek.

The spillway discharge channel receives its water through nine openings regulated by pivot-gates. The aggregate effective length of the openings is 67.5 ft. These openings are separated from one another by 7-in. reinforced concrete partitions set at an angle of 45° with the axis of the spillway sill wall, and making an angle of 27° with the line of flow through the spillway discharge channel. Abrupt deflections of stream lines are thus avoided, and the capacity of the discharge channel is correspondingly enhanced.

The spillway gates hang on vertical pivots, which are slightly to one side of the center of the gate. The greater water pressure on one leaf of the gate than on the other, to the extent of the pivotal eccentricity, is used to make the gates self-closing and to keep them closed as long as the water level is below the danger mark. The automatic opening of the gates is accomplished by carrying the shorter leaf to a greater height than the longer one. A rise of water above the lower leaf will reverse the previous balance of pressure, and the gates will open automatically.

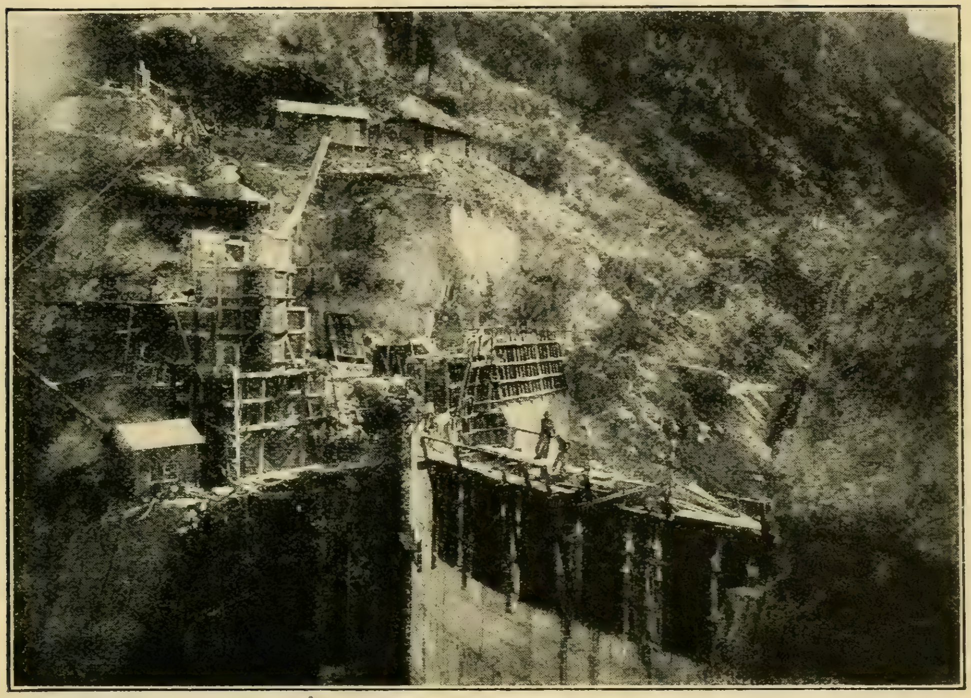


Fig. 8.—A Quiet Day. Laborers Scarce.

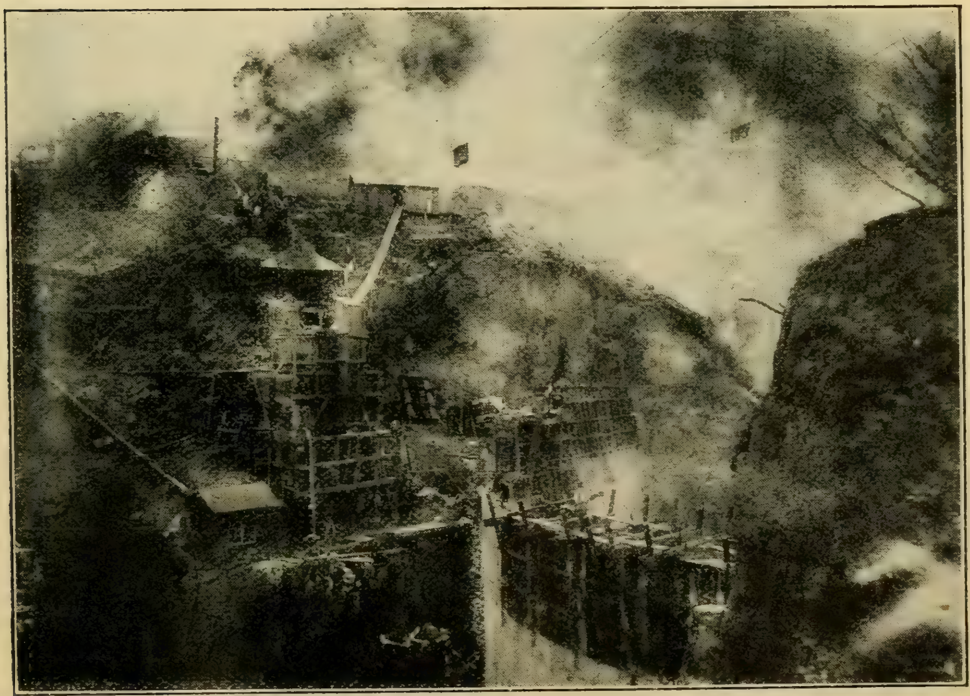
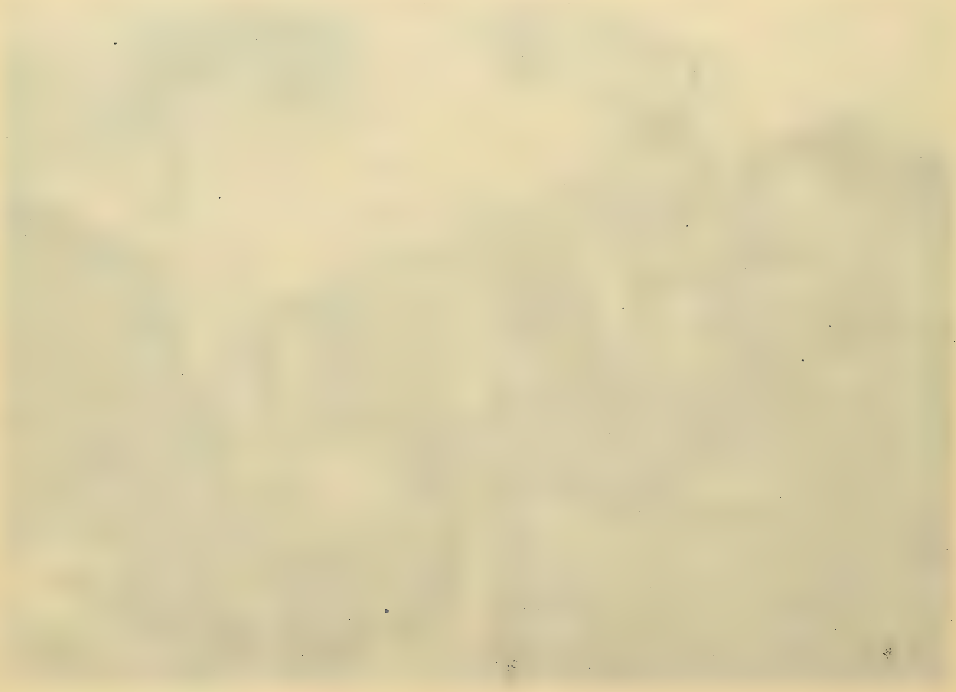


Fig. 9.—A Busy Day.



Outlet System.

The discharge of water from the reservoir is effected by pipes controlled by flap-valves outside and standard screw-valves inside a concrete outlet tower, Fig. 10. The intake ends of the pipes are protected against trash by screens movable for cleaning.

The short bridge connecting the top of the tower with the dam is not connected to it rigidly. This leaves the tower free for independent movement in the event of an earthquake.

The gate-tower is a continuation of a shaft in the rock formation, from the bottom of which a pipe tunnel, in rock, runs out under the dam. The pipes consist of an 8-in. service pipe, a 12-in. blow-off, and a 10-in. drain. The tunnel is refilled with concrete in which the 12-in. and 10-in. pipes are embedded, together with an additional 10-in. pipe through which runs the 8-in. service pipe. This arrangement makes the service pipe removable for repairs or renewal. Should the other pipes rust out, the holes left through the concrete will answer all purposes.

The lower 8-in. intake pipe and the 12-in. blow-off are carried through a short tunnel into the tower-shaft. These pipes are of $\frac{3}{4}$-in. cast iron, embedded in the concrete of the tower. All these pipes are provided with the usual collars or flanges to stop seepage.

Cement and Sand.

The cement used was furnished, under the Standard Specifications of the American Society of Civil Engineers for Portland Cement, by the Southwestern Portland Cement Company, El Paso, Tex.

Tests at the dam, of neat cement, and cement and sand briquettes, gave the minima shown in Table 3.

The appearance and feel of the Huacal sand were not reassuring, but as the use of the local sand, if possible, meant a material reduction in cost, and as the cement and sand briquette tests promised well, a trial of the Huacal sand was made in a block of concrete containing some 80 cu. yd. The results were anything but satisfactory. After 10 days of setting a pick could be driven 1 in. into the mass without any great muscular energy. The total mass was removed, and the hope of using local sand was abandoned.

Good sand is always more or less scarce in arid regions, a fact largely attributable to insufficient natural grinding and washing to destroy and remove the softer particles. This is particularly true of locations such

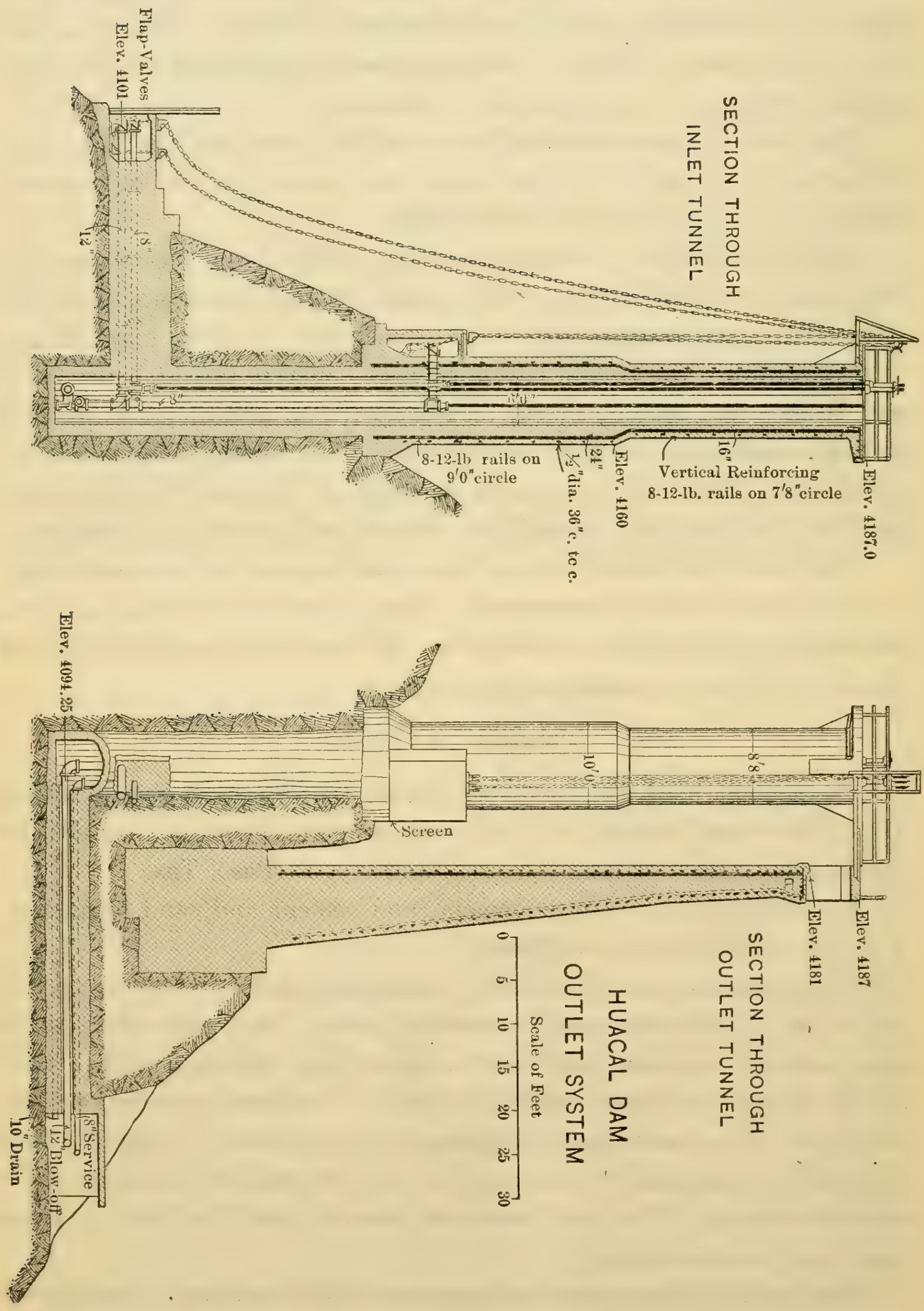

SECTION THROUGH
INLET TUNNEL
Flap-Valves
Elev. 4101
12"
8"
8"
8'0"
8-12-lb rails on
9'0"circle
24"
½"dia. 36"c. to c.
Elev. 4160
16"
Vertical Reinforcing
8-12-lb. rails on 7'8"circle
Elev. 4187.0
Elev. 4094.25
10'0"
8'8"
Screen
Elev. 4181
Elev. 4187
SECTION THROUGH
OUTLET TUNNEL
HUACAL DAM
OUTLET SYSTEM
Scale of Feet
0 5 10 15 20 25 30
8" Service
12" Blow-off
10" Drain

FIG. 10.

as the one in question, where the greatest length from the dam site to the basin crest is 5 miles, a distance far too short to expose the rock detritus, in its journey along the various stream beds, to the scouring and abrading influences requisite to making good sand, however suitable the parent rock might be.

TABLE 3.—Tensile Strength of Cement, in Pounds per Square Inch.

Neat Cement Briquettes.

Age of briquette, in days	1	3	4	5	6
Cement taken at random from store	290	485	685	660	640
Cement taken at random from mixing platforms.	210	....	....	675	790
Initial set: 2 hours 10 min. to 2 hours 30 min.					

1 Cement to 3 Sand Briquettes.

Description of Sand :					
Standard test	....	215	....	300	310
Nacozari, unwashed	....	....	88	100	....
Huacal, unwashed	....	180	....	130	235
Nacozari, washed	....	....	....	150	....
Huacal, washed	....	95	....	155	165
Huacal, washed (on 200-screen)	....	....	160	190	150
Crusher-run screenings, ¼-in. mesh	....	....	155	....	....
Mixed Sands :					
3 Nacozari, 1 Huacal, washed	....	....	....	100	....
1 " 1 " unwashed	....	....	106	....	....
4 " 1 " "	....	....	75	....	....
{ 1 " unwashed and } { 1 crusher-run screenings, ¼-in. mesh }	....	....	125	....	....

Concrete.

The writer's previous experience having shown a 1 to 2 mortar to be safe for water-tight concrete, and that a 1 to 2½ mixture bordered on the pervious, a proportion of 1 of cement to 2¼ of sand was adopted, giving an excess of about 25% of cement over the voids in the sand. The coarse aggregate was crusher run, graded from fragments of ¾ to 1 in. down to a coarse, sharp sand. The average of numerous tests gave the void spaces of the crusher-run aggregate, after compaction by shaking and jarring, as 38 per cent.

A series of tests was conducted to determine the proportion of sand to crusher run which would give a mixture of greatest density. These tests were made by progressive weighing of a box of known volume filled with the mix to be tested, the box being shaken and jarred during the process of filling. The greatest average weight was attained with 1 of sand to 2.03 of crusher run, the void space being 19 per cent.

The proportions actually fed to the mixers averaged 1 cement: 2 sand: 3.85 crusher run. The sand of the crusher run added to the

straight sand made the desired 1 to 2¼ mortar. The proportion of sand in the crusher run varied, of course, from time to time, with the wear and changing and consequent slacking and tightening of the crusher jaws. The foreman watched the composition of the crusher run closely, and from his observations adjusted from time to time the sand component so as to maintain practically constant the desired ratios.

TABLE 4.—WEIGHTS OF SAND AND ROCK.

Kind of sand.	Specific gravity.	Pounds per cubic foot.
Nacozari, slightly damp, loose	....	105.2
Huacal " " "	....	101.8
Crusher-run, loose	....	104.8
Nacozari	2.51	157.0
Huacal	2.43	152.0
Andesitic rock fed to crusher	2.65	166.0

TABLE 5.—SCREEN ANALYSIS OF SAND.

Screen No.	PERCENTAGE RETAINED.		PERCENTAGE PASSED.	
	Nacozari.	Huacal.	Nacozari.	Huacal.
20	71	60	29	40
30	12	21	17	19
40	5	8	12	11
60	5	4	7	7
80	2	1	5	6
100	1	2	4	4

The resultant mix gave a mortar approximately from 35 to 40% in excess of the voids in the coarse aggregate. Density of concrete was obtained with a sufficient surplus of mortar to coat thoroughly the surfaces of the 20-lb. to 300-lb. "plum" stones which were introduced as liberally as practicable. The "plums", first well wetted, were dropped into the mush concrete and worked around to insure liberation of any imprisoned air. The "plums" constitute probably 15% of the entire mass of concrete; a larger percentage would have been added if the hoisting facilities had permitted. The consumption of cement, as a whole, averaged about 1.3 bbl. per cu. yd. of completed concrete. Samples of finished concrete broken out of the actual structure weighed, dry, 152 lb. per cu. ft.

The concrete has proved to be water-tight to a marked degree, such slight dampness as appears in spots on the back face of the dam

being more probably due to seepage at over-night joints between one day's work and another than to percolation through the body of the concrete.

Plant.

The concrete plant was erected on the east side of the canyon at a higher elevation than the top of the dam. Cement was unloaded directly from the pack animals into store at the highest point of the plant, and the transportation was paid for by count of the sacks delivered. Sand, with transportation paid for by delivered weight, was unloaded and weighed into a bin near the bottom of the canyon and hoisted by bucket on inclined cable and delivered into gravity bins. The rock crusher was operated at an elevation corresponding with the spillway floor, and fed by rock from the spillway excavation. The product of the crusher was raised by belt and bucket elevator to the bins.

Cement, sand, and crushed rock gravitated from their respective bins to the batch-measuring hoppers, and thence to two mixers, one with a capacity of 12.7 cu. ft. to the batch, and the other of 21.7 cu. ft. The average composition of the mix was 1 cement: 2 sand: 3.85 crusher run.

The mixers delivered into a 1½-cu. yd. bucket slung between two standing cables, one crossing the canyon immediately below the dam and the other immediately above. Traveling carriages, one on each cable, moved in and out together. Hoisting tackle, independently operated, reached from each traveling carriage to the bucket bail. This arrangement permitted the depositing at will of the bucket at any point on the dam. Traveling and hoisting ropes were operated at speeds of about 600 ft. per min. light and 400 ft. per min. loaded. The maximum quantity of concrete placed in any one day was 114 cu. yd., and the average per day for the days on which the plant was running was 37.5 cu. yd. The minimum working time of mixing and placing 1 cu. yd. of concrete was 3.2 min., the maximum was 19.25 min. per cu. yd. (refilling outlet tunnels), with an average working time of 8.7 min. per cu. yd. for the whole work.

The machinery, picked up around the company's mines, and not without some incursions into the scrap pile, was of a heterogeneous character, incompatible with compactness or efficiency of operation. A saving in plant investment was effected at the expense of the constructional cost of the dam.

There were five steam boilers, ranging from 10 to 35 h. p. supplied with feed-water by a direct-acting, duplex, steam pump, drawing from water impounded above the dam. Six engines were used, a 25-h. p. for hoisting and a 30-h. p. for traversing the carriages on the cableways, a 26-h. p. on the rock crusher, a 30-h. p. on the sand hoist, and two 6-h. p. engines driving the concrete mixers. The fuel supply was chiefly wood from the live oaks cleared from the reservoir, liberally helped out with coal to compensate for the wood being green.

It would have been feasible to operate by electric current transmitted from the copper company's generating plant at Nacozari. This method was contemplated originally, but at the time was not deemed best. The experience of the work showed that the elimination of boiler and enginemen troubles and the elimination of transportation of coal, and other economies which would have followed with electricity as the motive power, would have resulted in a lower cost of construction.

The work was carried on during the Madero Revolution, when the country was in a state of turmoil; near-by collisions between the opposing forces were frequent, and the labor supply was disturbed and uncertain.

Forms.

The passing of any bolts, wire, or other form fastening, through from face to face of the dam was prohibited. The lower portion of the dam was constructed with wood forms of the customary stud and sheeting type, to a height above which the external struts bracing the studding would have become inconveniently long. The upper 60 ft. of the dam was built with steel forms (Fig. 11). These were in units 9 ft. wide for the water face and 8 ft. 3 in. for the back face, and 12 ft. high. They were generally used in threes, making a block space for pouring concrete 27 ft. by 12 ft. and of the width of the dam.

No attempt was made to vary the radius of the back face forms to conform to increasing radius due to decreasing thickness of dam with height. The forms were shaped to a compromise radius. The slight angle made by the adjacent forms, where not strictly conforming to the true radius, was imperceptible on the finished work. The location of the dam and its commercial use did not warrant the increased cost of forms of variable radius which would have been necessary for precision of shape. The front face of the dam being vertical,

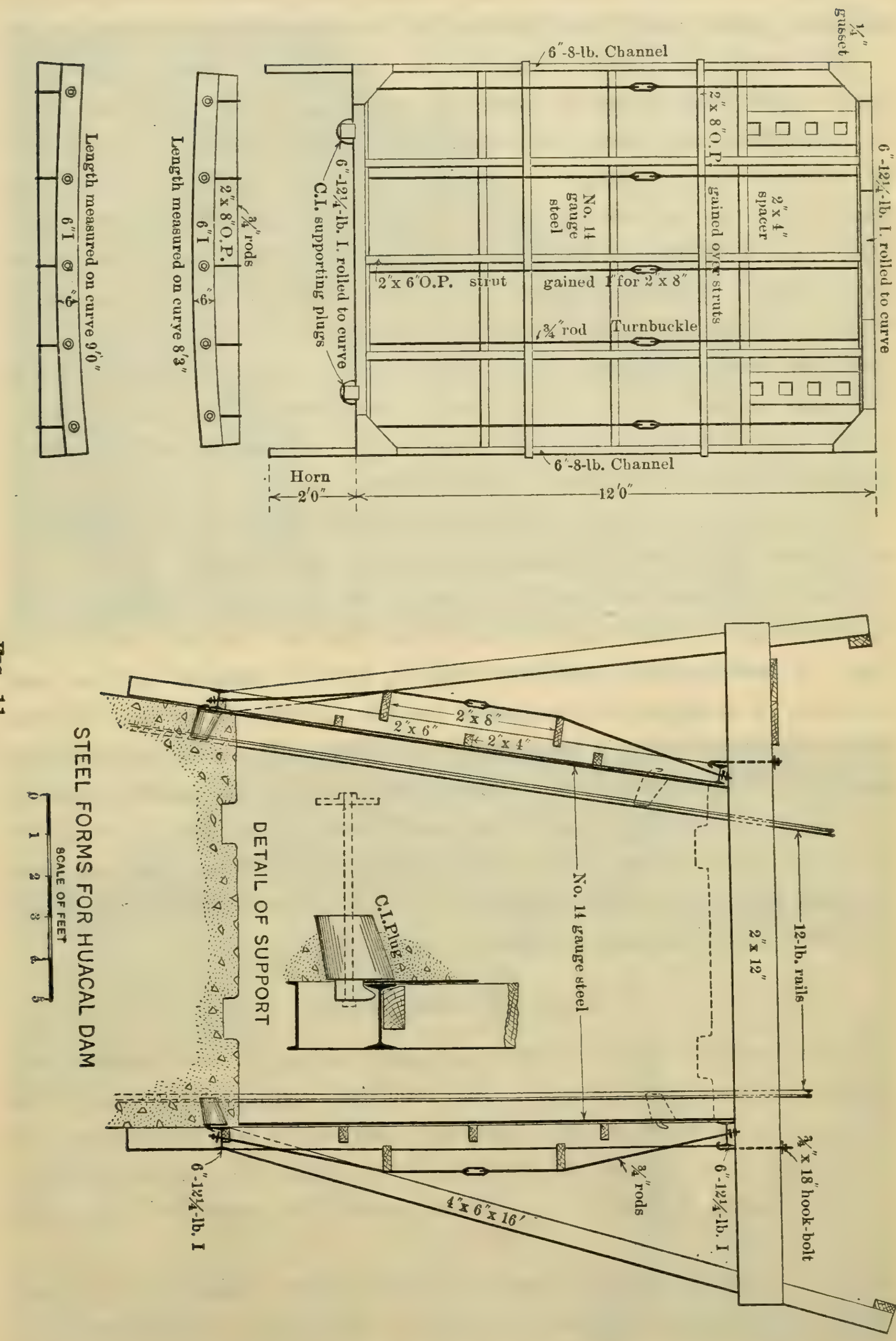


FIG. 11.

STEEL FORMS FOR HUACAL DAM

its radius is constant. The front forms, being first set accurately to position, constituted a base from which were set the back forms.

The steel forms were supported by their lower edges engaging in hook-shaped projections on taper cast-iron plugs built in advance into the concrete as the work progressed. The plugs retained their position by virtue of being inclined downward into the concrete. After use, the plugs were withdrawn to be used again and the pockets were filled with concrete. Two plugs were used to a form unit, one under each corner. The plan proved satisfactory, except in some instances where the concrete bucket with its load struck heavily the top of the form and jarred the plugs loose. However, no instance occurred where the plugs actually let go. A bolt might be added to the plug, as shown by the broken lines on Fig. 11, which could be removed with the plug, the washer alone being lost in the concrete.

The walkway on top of the forms fulfilled admirably its purpose of a safe and convenient working platform for the concrete gang.

The forms were handled into place by the overhead cableways. The cost and time of removing the steel forms from set concrete and placing them in new position was low and in marked contrast to the cost and time of stripping the ordinary wooden forms. After the concrete work was completed, the steel forms were dismantled and part of their structural shapes and turn-buckles were used for the spillway gates. The remainder of the metal work was found useful, and absorbed for odd jobs by the mine mechanical department. There was but little material sent to the scrap pile, whereas with wooden forms there would have been practically no salvage.

Metal Reinforcements.

Both faces of the dam are reinforced vertically and horizontally against temperature stresses. The vertical members are old 12-lb. mining rails, 4 ft. 6 in. from center to center on the front face, and on the back face 4 ft. 6 in. from center to center for the lower 35 ft. of their length, and 9 ft. 0 in. from center to center for the upper 26 ft.

The horizontal reinforcement consists of five rings of 12-lb. rail immediately below the crest of the dam, and, from there down, ⅝-in. round and square rods, with spacing varying from 15 to 18 in. from center to center.

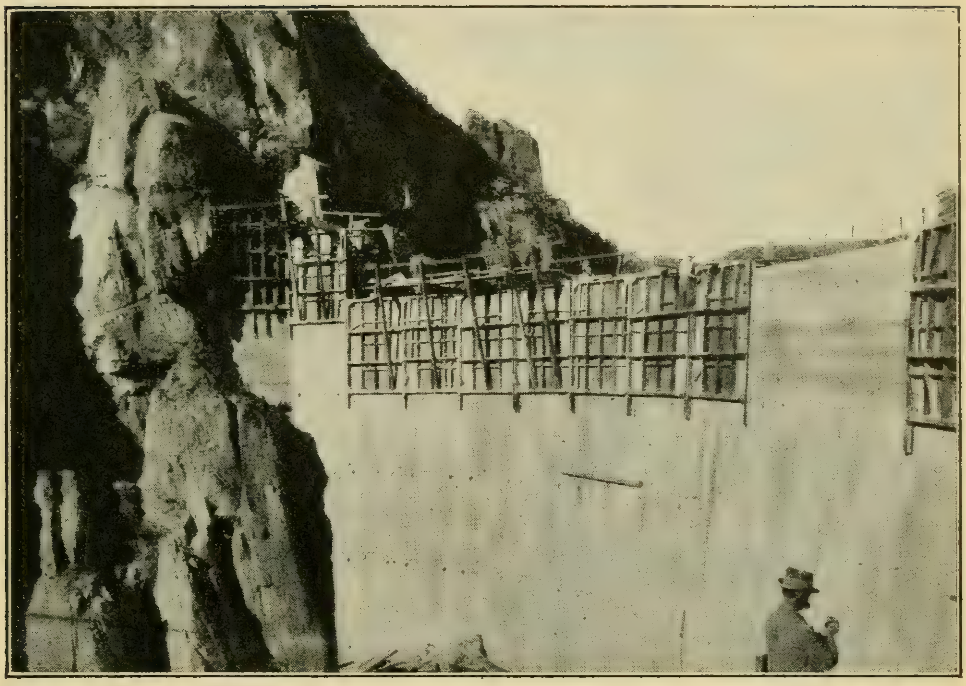


FIG. 12.—REAR FACE OF DAM, SHOWING STEEL FORMS IN USE.

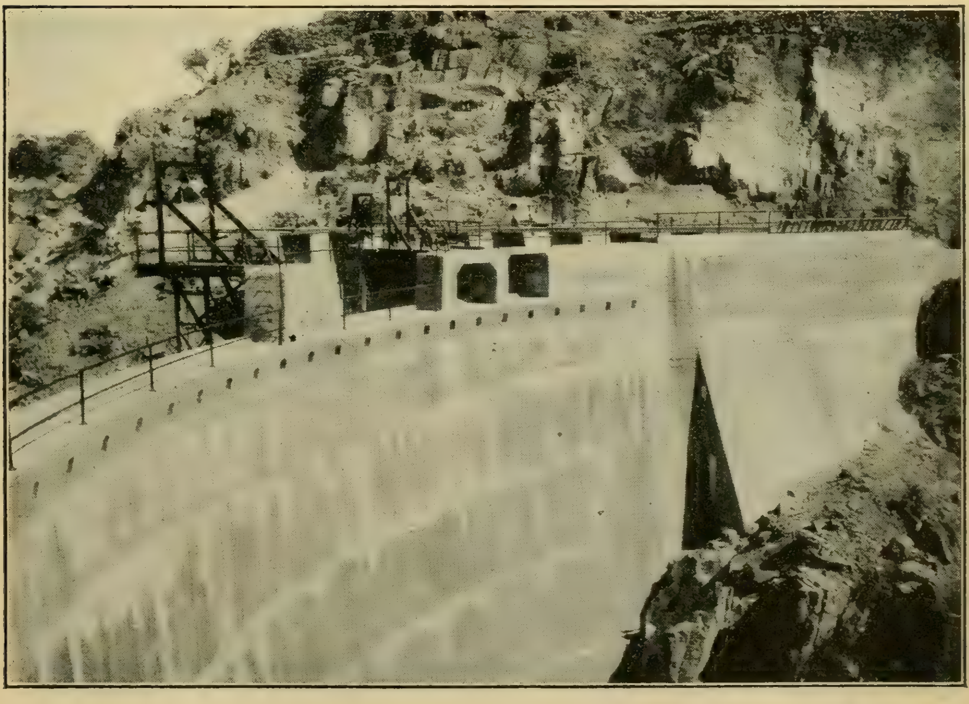


FIG. 13.—UPPER PORTION OF DAM, REAR FACE. GRAVITY SECTION ABUTMENT AT EAST END.

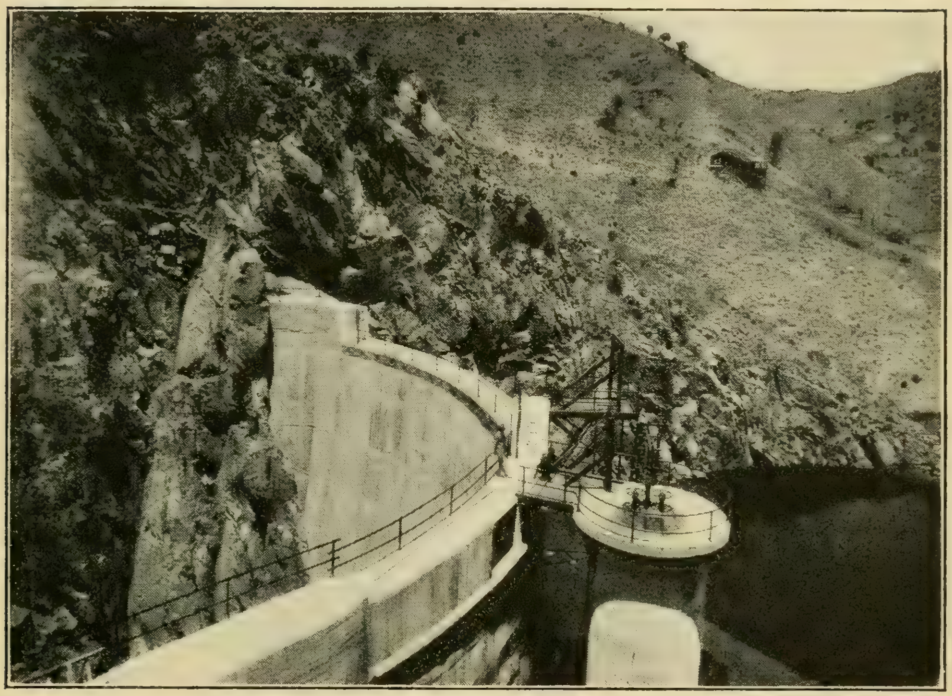


FIG. 14.—TOP OF DAM, FROM SPILLWAY.

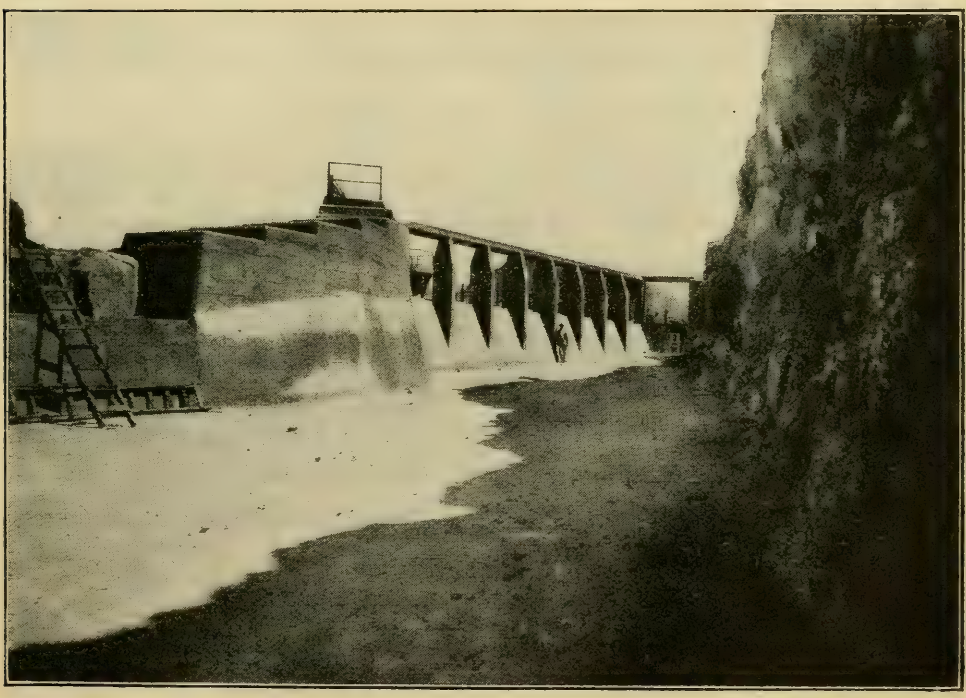


FIG. 15.—SPILLWAY. GATES NOT IN PLACE.

The lower 30 ft. of the dam is not reinforced. The length of the dam, measured on the arc, is comparatively small in this zone, and the possible temperature range is materially less than in the higher portions. Practically, the front face of the dam for this depth will be constantly immersed, which tends to uniform temperature. On the back face the lower part is less exposed to the sun than the upper.

TABLE 6.—CONCRETE QUANTITIES.

		Cubic yards.
Main dam and outlet tower		4 093
Spillway weir	475 cu. yd.	
Spillway floor and retaining wall	308 " "	783
Water-cushion dam		82
Miscellaneous		30
Total		4 988

PIPE LINES.

Water from the reservoir is conducted to the mill tanks at Nacozari through a pipe line, 2.91 miles long, composed of 1.13 miles of 8-in. and 1.78 miles of 10-in. steel riveted pipe, both of No. 14 gauge, furnished by the Lacy Manufacturing Company, of Los Angeles. Only on the lower 3 600 ft. of the line could wagons be used for the distribution of the pipe; for the remaining distance the pipe had to be packed by animals and men. Under these conditions, the savings to be made by using light riveted pipe outweighed considerations of permanency.

For wagon transportation, the pipe was made up in 20-ft. sections, and for pack animals in 10-ft. sections. Where buried in the ground, the usual California drive joints were used, and where exposed, flanged joints. The flanges are of pressed steel, riveted to the pipe. The flange dimensions were: for 10-in. pipe, 14¼ in. in diameter, 12¾-in. bolt circle, 12 ½-in. bolts; and, for 8-in. pipe, 12¼ in. in diameter, 10¾-in. bolt circle, 10 ½-in. bolts. When pipe had to be cut, or articulation was needed, bolted joints of the loose-ring type, with triangular rubber gaskets, were used. For water-tightness, the drive joints are dependent on the asphalt coating which the pipes receive in the final process of manufacture. In the field, after entering the end of the driven pipe, kerosene is poured around the outside of the joint and fired, and if necessary refired, until the asphalt coating softens and runs; then the

TABLE 7.—Cost of Dam.

				Per cubic yard of concrete.	
Roads and trails, labor	....	....	$1 528	$0.306	
Plant:					
Buildings, material	$2 985				
" labor	803	$3 788			
Sand bins, material	$970				
" labor	416	1 386			
Cableways. material	$1 574				
" labor	734	2 308			
Machinery and supplies. material	$4 363				
Installing machinery, labor	3 982				
Transporting machinery	1 190	9 535			
Water tanks		158	17 175	3.443	
Miscellaneous, unsegregated, material	$2 395				
" " labor	7 869		10 264	2.058	
Gates and control fittings, tower	$800				
Installing piping, tower, labor	880		1 680	0.337	
			$30 647		$6.144
Excavations:					
Outlet tunnel	$365				
Inlet tunnel	231				
Intake shaft	275	$871			
Spillway		2 264			
Foundation trenches, etc		2 173	5 308		1.064
			$35 955		$7.208
Concrete:					
Cement at Nacozari	$26 840				
Transportation of cement	2 847	$29 687		$5.952	
Sand, screening and handling	$3 007				
Transportation of sand	8 430	11 437		2.293	
Coarse aggregate, crushed rock from spillway excavation. Cost of crushing not segregated.					
Forms:					
Lumber	$4 712				
Steel	2 600				
Labor	6 151				
Transportation	1 189	14 652		2.937	
Mixing and placing concrete, and crushing rock, labor	$6 541				
Fuel for above, coal ... $1 318					
Transportation of coal ... 267					
Wood ... 2 048	3 633	10 174		2.040	
Blacksmithing, labor	$1 248				
" material	408	1 656		0.332	
Reinforcing material	....	1 454	69 060	0.291	$13.845
			$105 015		$21.053
Superintendence	....	....	2 790		0.559
Engineering, field	....	$2 815		$0.564	
" plans and supervision	....	3 670	6 485	0.736	1.300
Total	....	....	$114 290		$22.912

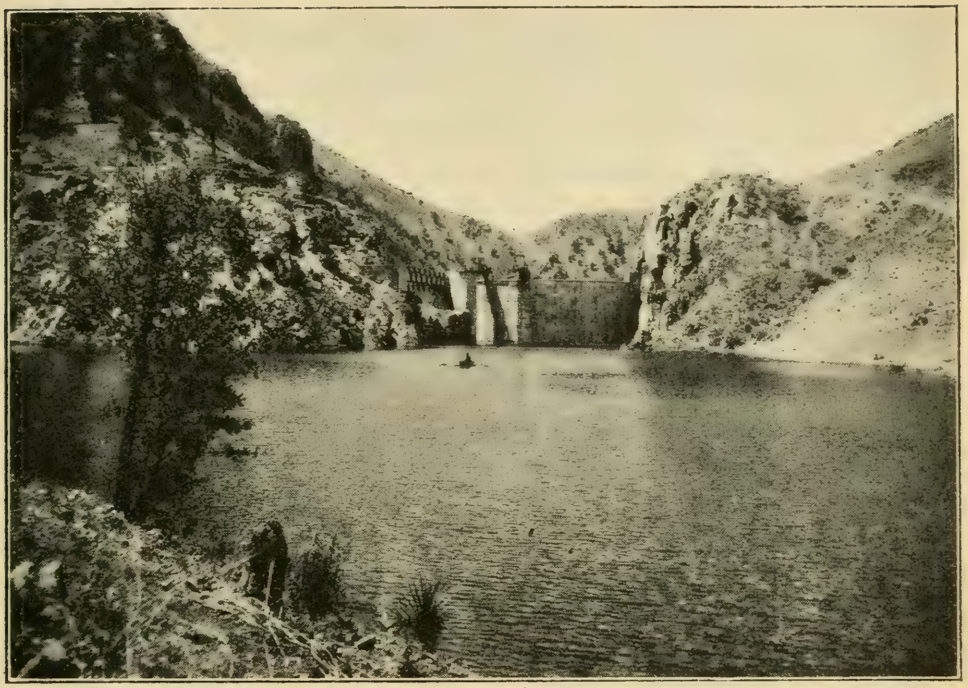


FIG. 16.—FRONT OF DAM, WITH 53 FEET OF WATER.

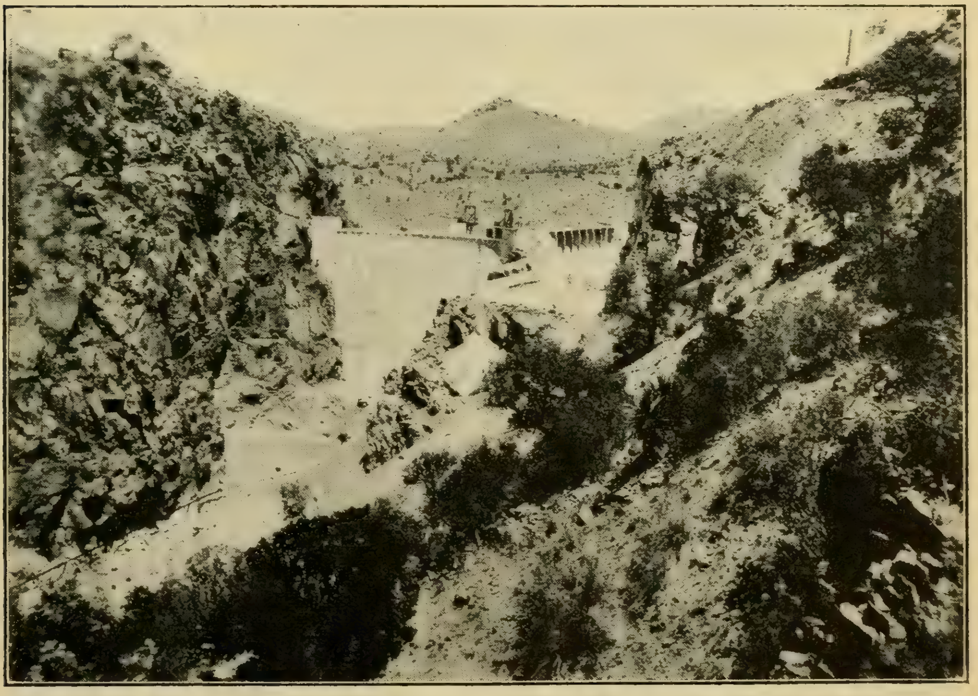


FIG. 17.—MAIN DAM, CUSHION DAM, AND SPILLWAY.

pipe is driven home. In straightaway work, a gang of experienced men can lay about 2 500 ft. of 8-in. or 2 200 ft. of 10-in. pipe in a day. When properly driven, and the pipe ends have not been deformed by transportation and handling, the percentage of leaky joints on testing out is very small. Leaky joints are remedied by galvanized-iron sleeves of sufficiently greater diameter than the pipe to leave an annular space of about $\frac{3}{4}$ in. into which neat cement or cement mixed with fine sand is rammed. When, from necessity, sleeves have to be put on a pipe under pressure, a closeable vent should be provided at the mid-length of the sleeve to discharge all leakage water until the cement sets. Seam leaks of minor nature are curable by floating fine sawdust through the pipe. With large seam leaks, it is preferable to put in a new section of pipe.

TABLE 8.—Cost of Pipe Line.

Huacal pipe line	5 974 ft. 8-in. pipe. 9 415 " 10- " "
Total length	15 389 ft.

		Total cost.	Average cost per linear foot.
Material:			
Pipe and fittings	$12 456		
Trestles	1 857		
Grading tools and supplies	561		
Transportation of material	790	$15 664	$1.02
Labor:			
Grading	$4 546		
Framing and erecting trestles	1 051		
Laying and covering pipe	2 056	7 653	0.49
		$23 317	$1.51
Superintendence		310	0.03
Engineering, field	$950		
" designing and supervision	816	1 766	0.11
Total	....	$25 393	$1.65

Break-pressure tanks of concrete, with "Davis" float-valve control, are introduced at two convenient points on the line, which limit the maximum static pressure to 179 ft. for the 8-in. and 283 ft. for the 10-in. pipe. At the summits, air relief and prevention of vacuum are afforded by 2-in. pipes carried up the mountain side to an elevation above the static head. The open ends of these pipes are protected by wire gauze to prevent the intrusion of foreign substances and living

things. Where the situation did not permit of open pipes, "Simplex" combined air relief and vacuum valves were used. The breakage of the pipe or the sudden opening of a blow-off valve at a low point in the line, thereby causing a partial vacuum, are to be particularly guarded against in light pipes. A vacuum which might be without ill effect in a heavy metal pipe line is absolutely fatal to thin riveted pipes, which have but little strength to resist pressure from without.

Test of Pipe Capacity.

Color tests of the velocity of the water traversing the pipe line were made by using permanganate of potash. Through 4 354 ft. of the 8-in. line, with a fall of 170 ft., the velocity was found to be 7.223 ft. per sec., from which the discharge was computed as 2.521 sec-ft.; the corresponding value of n in Kutter's formula is 0.0114. The velocity through 9 415 ft. of the 10-in. pipe, with a fall of 110 ft., was found to be 4.689 ft. per sec., and the computed discharge 2.555 sec-ft.; and the value of n is 0.0116. A volumetric test, made at the mill tank, gave the discharge as 2.529 sec-ft. In view of the fact that the pipes and tank were not calibrated, the three determinations of 2.521, 2.555, and 2.529 cu. ft. per sec. may be said to be in perfect agreement, the difference between the highest and lowest determination being but 1.33 per cent.

The writer takes this occasion to express his thanks to Mr. J. S. Williams, Jr., General Manager, and to other officers of the Moctezuma Copper Company, for their aid and co-operation in the work, and to Mr. C. A. Dodge for his effective supervision of the work of construction.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

By Harrison S. Taft, Esq.

To be Presented May 20th, 1914.

Synopsis.

During the past few years considerable discussion has taken place among engineers and chemists as to the feasibility of using cement in sea water, and the practicability and commercial success of reinforced concrete docks. In the United States the construction of reinforced concrete docks is in a very embryonic state, and the concrete of structures in sea water has not always been successful. On the other hand, concrete has been used successfully for more than 50 years in Europe for structures exposed to the action of salt water, and English engineers have been building reinforced concrete docks for about 20 years.

Part I of this paper contains a brief outline of the reinforced concrete dock problem as developed in Europe and Asia, the work in each country being examined. Part II reviews the art as carried out in the United States, both in sea water and fresh water. These two parts cover descriptions of reinforced concrete docks, as well as construction features of special interest.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Part III takes up the subject of failures in past uses of cement in sea water in America, points out the reasons therefor, and brings out the fundamental principles covering the successful use of concrete in structures exposed to salt water and frost action. A foot-note calls attention to the chemistry of salt water, cement, and the use of concrete in sea water, in order to bring out discussion on this phase of the problem.

Part IV goes briefly into the question of the initial cost of reinforced concrete docks, as well as the cost of annual repairs.

Part V is a review of opinions of foreign expert dock engineers on the use of concrete in sea water and on the practicability of reinforced concrete docks; there is also a short review of the use of concrete in foreign breakwaters, closing with a few remarks as to the reasons for the success of foreign engineers in using concrete in sea water.

Part VI contains the writer's conclusions, wherein he cites the relative positions of English and American engineers in respect to the reinforced concrete dock question, and a more general use of concrete. He also speaks of progressive engineers possessed with the determination and courage to develop new ideas and uses for concrete, with due consideration as to previous failures, while conservative engineers still cling to old-time, long-established practices.

In the closing paragraph the writer calls the engineer's attention to the necessity of special training and knowledge for the successful execution of concrete structures in salt water.

The object of the paper is to make known to the Engineering Profession the exact up-to-date situation with respect to the development of this problem, the world over, the feasibility of using cement in sea water, and the practicability of building reinforced concrete docks.

Introduction.

In undertaking a study of the reinforced concrete dock problem, the subject has so many phases as to require a number of chapters to cover fully all the various ramifications into which it leads. From a world point of view, reinforced concrete docks are not a novelty. On the other hand, their practicability and commercial economy have perhaps as yet to be proven, especially in America. It is true that there are many reinforced concrete docks in successful operation in foreign countries, the most extensive development being found in England. As

their size and number are steadily increasing, it would appear that this class of dock construction has proved its worth, and sooner or later will become the standard type where a structure of a permanent nature is desired.

As reinforced concrete docks are somewhat of a novelty in the United States, it is perhaps necessary for engineers to seek the experience of foreign countries in order to obtain the necessary data on which to base their conclusions as regards the feasibility or otherwise of this type. Thus it is eminently fitting to investigate first what has been done in various parts of the world in the way of building such docks and the success accompanying such undertakings, and then discuss other phases of the problem, as to what precautions, etc., are necessary to render them a commercial, practical, and engineering success.

Part I.—Reinforced Concrete Docks in Foreign Harbors.

England.

In building reinforced concrete docks and other concrete structures in sea or fresh water, it is only natural that a forested country should be the last to take up the development of such a material. Consequently, America has been far in the rear in regard to this question, as compared with what other and older countries have accomplished. In England, France, Germany, Italy, and other European countries, in Australia, and in Asia, as well as in certain South American countries, concrete sea-walls, breakwaters, dry docks, piers, trestles, coaling stations, etc., in salt water, have been in existence for years. In looking up past undertakings along these lines, the writer was amazed to learn of the large number of reinforced concrete docks that have been built in foreign countries and the great success that has apparently been attained with them.

Southampton.—One of the most noted developments in reinforced concrete construction, as applied to harbor and dock development, is that of the London and Southwestern Railway Terminals at Southampton, England. The first and most prominent reinforced concrete structure in connection with this terminal was a coal barge jetty, 300 ft. long and 20 ft. wide, built in 1904, on the Hennebique system of driven concrete piles. The piles are about 44 ft. long, standing in 29 ft. of water at high tide, the rise and fall of the tide being about 13 ft. Each pile carries a maximum load of 17 tons.

This structure carries a very heavy traveling coal-hoisting apparatus for unloading coal from large vessels docked on one side, into harbor barges or scows which lie on the other side. Thus the jetty is subjected to constant blows from both sides, in addition to the heavy vibration due to the traveling machinery it supports.

In speaking of this jetty, Mr. Francis Wentworth-Shields, Dock Engineer for the London and Southwestern Railroad, says:

"Though the impacts from the vessels and scows cause this whole jetty to sway, there seem to have been no signs at the end of the first $2\frac{1}{2}$ years of its existence of any of the concrete peeling off."

During recent years this dock or jetty is said to have shown some signs of deterioration, due to the vibration of the heavy machinery traveling along it, the supposition being that it was built too light in the first place to absorb the heavy vibrations to which it has been subjected. Still, the dock is said to have considerable elastictity. In one instance it was in heavy collision with a steamer, two piles and the beams they carried being broken. The dock was effectively repaired, but perhaps with some difficulty, though it is claimed that the repairs were easily accomplished.

The same railroad company has built several other reinforced concrete pile docks at Southampton, designed to carry the same heavy deck loads as the coaling jetty, but without the heavy vibration to which this jetty is subjected. Though built on the same system of construction as the coal jetty, the latter docks have shown no signs of wear or deterioration. It is said, on the best of authority, that they have cost nothing to date for maintenance.

The largest of these is the extension, on the Itchen Front, of "The Empress Dock", a widening of the so-called "Old Extension Quay" by a reinforced concrete pile structure, 50 ft. wide and about 1 300 ft. long, parallel to and securely dovetailed into the old quay wall. This widening dock is built of complete concrete bents, along the tops of which are steel deck-beams or stringers, which in turn are covered with 4 in. of wood, a wooden block pavement being used for the wearing surface. The depth of water at low tide at the face of this structure is 35 ft., which, with a 13-ft. rise and fall of the tide, gives a depth of 48 ft. at high tide.

One of the finest cold storage and cattle stations in existence was built at Southampton in 1905 to accommodate the foreign cattle trade.

The landing stage or jetty of this station is a reinforced concrete structure, 200 ft. long and 38 ft. wide, connected with the main land by two runways, 142 ft. long and 15 ft. wide.

On the opposite side of Southampton Harbor, at Woolston, on the Itchen, there is a reinforced concrete landing dock, 136 ft. long and 100 ft. wide, built in 1899 on the Hennebique system. This was the pioneer of reinforced concrete dock construction in Southampton waters. Up to date, this dock has cost practically nothing for repairs, except for damages due to the fact that it was rammed or otherwise damaged by a large steamer; it is in excellent condition at present. The cost of making the repairs is said to have been very small.

Up to the present time, it appears that at least six reinforced concrete docks, jetties, or quays, have been built in Southampton waters.

In building one of the Southampton docks, it is stated that some of the concrete piles were sprung out of line in driving them. A prominent American engineer reports that he saw a number of piles from 1 to 2 ft. out of line, but that they showed no signs of cracks. It has been stated that, in handling Chenoweth concrete piles, up to a length of 61 ft., and 13 in. in diameter, they were rolled about like wooden piles, at times having quite a spring in them. Under this treatment they showed a remarkable degree of elasticity and no signs of cracking.

Bending of Piles.—In discussing the bending of concrete piles, it is of interest to note a series of tests made on a hollow telephone pole in Fulham, London, England, in 1911. The pole was 44½ ft. long, 17 by 17 in. at its base (outside dimensions), tapering to 8 by 8 in. at its head. The thickness of the shell was 2 in., making the inside dimensions of the pole 13 by 13 in. at the base and 4 by 4 in. at the top. The vertical reinforcement consisted of 248 $\frac{3}{16}$-in. rods of high-tension steel, the ultimate tensional stress being from 80 000 to 85 000 lb.; 56 rods were grouped at each corner, 6 rods being spaced evenly on each of the four sides of the pole. The area of the concrete at the base was 106 sq. in., and the area of the steel 6.85 sq. in., a ratio of 0.0445. In making the test, the pole was set in 5½ ft. of massed concrete, with the pulling rope attached to its upper end.

In Table 1 is recorded the pull, in pounds, the deflection, and the permanent set after the loads had been released.

After applying the 6 000 lb., the test was discontinued for 3 days, and then the load was slowly increased to 9 200 lb., which gave a de-

flection of 66 in. and a permanent set of 21 in., with "cracks on tension side and permanent cracks more numerous and pronounced." There was no sign of failure on the compression side of the pole.

TABLE 1.—TEST OF A HOLLOW CONCRETE POLE AT FULHAM, ENGLAND.

Loads, in pounds.	Deflection, in inches.	Permanent set, in inches.
300	3/4	None that could be observed.
500	2	
1 000	4 1/4	
1 250	5	
1 500	6 1/4	
1 750	8	
2 000	9	
2 250	10 1/8	
2 500	12	1/4
2 750	14	1/2
3 000	14 3/4	5/8
3 550	19 1/8	3/4
3 100	16 1/2	1/2
3 500	18 3/4	5/8
4 000	22	3/4
4 500	25 1/8	1 1/8
5 000	29	1 3/4
Load gradually increased to 5 750	34	Pulling wire broke. Pole flew back, vibrated short time and came to rest in vertical position with no permanent set.
Load gradually worked up to 6 000	37 1/8	2 in. immediately after release of load. After interval of 1 1/2 hours, set was reduced to 11/16 in.

Add to the above loads 100 lb. for weight of wire and recording apparatus, etc.

Notes on Behavior of Pile.—At 3 350 lb., slight hair cracks appeared on tension side at about 5 ft. 6 in. above foundation.

At 3 600 lb., slight hair cracks appeared at regular intervals about 6 to 9 in. apart.

At 4 100 lb., slight shear cracks appeared, starting at 33 in. above foundation and extending vertically, at a distance of 2 in. from tension side, for some 3 1/2 ft. in length.

At 4 600 lb., hair cracks occurred regularly, 3 or 4 in. apart on tension side.

At 5 100 lb., cracks on tension side were noticed to be traveling across the sides of the pole to within 6 in. of compression surface.

At 6 100 lb., the shear cracks were more pronounced. The hair cracks on tension side were about 1 in. apart, but no signs of failure appeared on compression side.

The load was again applied. When a deflection of 73 in. was obtained, signs of failure appeared for the first time on the compression side, and the pole failed, but no record of the amount of the pull was obtained. The final examination showed that the pile failed equally for its entire length on the shear and tension sides, but without any local weakness. The compression side failed from the base up for a distance of about 2 ft. The bending moment at the base of the pile was reported to be 4 863 936 in-lb., a most remarkable test, which seems to substantiate the experience quoted above.

Liverpool.—The Mersey Docks and Harbor Board appears to have made a very limited use of reinforced concrete in its latest port develop-

ments, in spite of the extensive tidal and graving docks of concrete constructed at Liverpool. As a matter of fact, most of its first so-called reinforced concrete docks were in reality semi-concrete structures, that is, wooden piles carrying a concrete deck system. The oldest of these structures, the Cattle Wharf, Prince's Stage, was built in 1899-1901 on greenheart piles, with the usual Hennebique system of concrete deck-beams and slab. In addition to the Cattle Wharf, there are two other dock-quays in Liverpool, built on the same system, designed for a load of 3 tons per sq. yd., with a test load of 4½ tons.

Another of Liverpool's reinforced concrete docks is the Prince's Dock, West Quay, completed in 1905. This structure was designed to carry a super-load of 3 tons per sq. yd., or about 66 tons per pile. The piles are of concrete, 16 in. square, spaced 15 ft. 9 in. from center to center, across the dock, and 12 ft. 6 in. from center to center, longitudinally, and resting on rock.

At the west end of Liverpool's dock system is the so-called Brocklebank Dock, 226 ft. long and 64 ft. wide, a reinforced structure built in 1908 on the Hennebique system, and designed to carry the same loading as the Prince's Dock. The test load applied was 9 tons per sq. yd. The piles of this dock are 20 in. square, and had to sustain a load of about 95 tons each, when the test load was applied.

During the first year of its existence there was considerable trouble with the Cattle Wharf and other semi-concrete structures, due to the permeability of the deck-slab part of the structure. The dampness from the salt water entered the permeable concrete and attacked the steel reinforcement on the under side of the slab, causing pieces of concrete to fall off. The defect was overcome by applying a heavy coating of cement to the under side. The upper side of the deck-slab, which is washed daily, shows no signs of deterioration. This emphasizes the fact that great care must be taken in placing concrete in structures standing over or in salt water; there must be the closest inspection during their construction. As a result of this trouble, it appears that the steel reinforcement must be kept farther from the surface of the concrete in salt water concrete structures than in ordinary work. Whereas, at first a minimum of 1½ in. was supposed to be sufficient, the engineers of the London and Southwestern Railway have found this to be insufficient, and are now specifying a minimum of 2 in. for their new structures. Another difficulty encountered was the nu-

merous joints in the Liverpool system of dock construction. This appears to have been overcome by cleaning out the joints and filling them with a rich cement grout, with such success that the docks are reported to be in very satisfactory condition at the present time.

Thames River, Etc.—Within the bounds of the "Port of London Authority", and at other places on the Thames, many reinforced concrete docks or quays have been constructed. One of the most prominent of these is the Thames Haven Jetty Head, near the mouth of the river. This structure was built in 1908, for the berthing of large vessels, and is 136 ft. long and 32 ft. wide. It is supported by 19 columns or piers, 5 ft. in diameter up to low-water mark but of oval shape, 5 by 2½ ft. above. Each column rests on two 15-in. concrete piles of octagonal section, 45 ft. long, with a penetration of from 18 to 20 ft. After these concrete piles had been driven to refusal, a temporary iron caisson was put down over them. The reinforcing hoops were then lowered into place around the two piles and the caisson was filled with concrete. The caisson was afterward removed, being built in half sections.

Farther up the Thames, 20 miles from its mouth, is the Purfleet coaling jetty, built in 1904, a reinforced concrete structure, 250 ft. long and 34 ft. wide, carrying two heavy traveling cranes. The concrete piles are 14 in. square, from 40 to 50 ft. long, with from 15 to 18 ft. of penetration. The deck-slab is 5 in. thick. The jetty is connected with the mainland by a reinforced concrete approach, over which cars are run to the jetty itself. On one occasion this jetty was rammed by an 8 000-ton steamer, eight piles and 20 ft. of the decking being damaged. That the damage was not more extensive is said to have been due to the firmness of the horizontal decking. In repairing the dock it was necessary to withdraw the eight broken piles and drive new ones. In conducting this repair work, it was especially noticed that the steel reinforcing bars of the original structure showed no signs of rust, though it is well to note that the water at this point of the Thames may be brackish, and not real sea water. The repairs were efficiently made, and the quay is still doing duty.

Another interesting reinforced concrete coaling dock is at Dagenham-Essex-on-the-Thames, 10 miles below London, built in 1901. It is 780 ft. long and 35 ft. wide, and carries eight heavy traveling cranes, weighing 60 tons each, besides a railroad track. Each supporting col-

umn consists of three concrete piles encased in a concrete cylinder filled with concrete. Though it may seem a trifle odd, a careful study of photographs of this structure indicates that the tops of the columns are made with "capitals"—a well-proportioned and artistic structure as regards concrete dock work. A similar structure, known as the Prince of Wales Pier, was built in 1903 at Falmouth.

The Thames Iron Works and Shipbuilding Company has constructed at Dagenham a very substantial reinforced concrete dock, designed to carry a concentrated load of 60 tons, for the berthing and fitting out of dreadnaught battleships. In the construction of this dock, the Williams type of concrete piles was used for the first time. This type consists of an I-beam surrounded with $\frac{3}{16}$-in. wire, 12 in. from center to center, the whole being encased in concrete, with special provision at each end to take up the reaction of the driving.

In the same vicinity is the Hornchunk Dock and approaches, built in 1906. The main structure is 400 ft. long and 24 ft. wide, the approach being 280 ft. long and 16 ft. wide; both were built on the Hennebique system. The concrete pile bents are 14 ft. from center to center, braced diagonally and support a concrete beam and deck-slab system. Along the water-front side of this dock and its approach, a concrete curtain-wall was built between the piles. This curtain-wall in turn is protected by a wooden fender system. The plans seem to indicate that this structure was designed to carry a 27-ton crane in addition to a 27-ton locomotive.

A reinforced concrete coal dock was built in 1906 at Rochester, on "The Medway", a river flowing into the estuary of the Thames. This dock consists of a main water-front section, 32½ ft. wide and 340 ft. long, connected with the land by two concrete approaches, 100 ft. and 180 ft. in length, respectively. It was built on the Hennebique system, and carries a heavy traveling crane in addition to two railroad tracks. This was one of the first large undertakings in reinforced concrete dock construction on the Thames or its tributaries. At the time it was built it was looked on as the most important structure of its kind in England.

Though the exact number of reinforced concrete docks now in existence on the Thames or its tributaries is perhaps unattainable, it appears that they are more numerous in those waters than in any other part of England.

General.—Perhaps the largest reinforced concrete dock constructed up to date in England is at Swansea, on the southwest coast of Wales. It is a Hennebique design, almost 2 000 ft. long, completed in 1908, the whole structure being built in the dry. Some of its columns are 3½ ft. square. As this structure is used as a coal loading quay, and stands in 40 ft. of water, it is subjected to heavy loads and severe lateral shocks.

At the naval dock yard at Rosyth, Firth of Forth, Scotland, is found one of the most unique and interesting reinforced concrete docks or piers in existence. It is triangular in shape, 620 ft. long, has been finished recently, and forms the entrance pier to a tidal basin. The outer end consists of seven concrete monoliths with a mass concrete fill in the center, making a solid concrete head, 127 ft. long and 65 ft. wide, covering an area of some 8 300 sq. ft. The rest of the pier consists of twenty concrete monoliths, 25 by 30 ft., supporting a reinforced concrete structure consisting of girders, columns, arches, braces, and deck-beams. The upper decking consists of 3 by 10-in. creosoted wooden joists covered with a creosoted planking arranged so as to prevent any water from collecting thereon. It is a most massive and homogeneous structure, well capable of absorbing any heavy stresses it may receive from warships lying alongside.

In the Bay of Bristol, at Clevedon, where the tide has a rise and fall of some 49 ft., there is another interesting reinforced concrete structure, in the nature of a landing stage. It is 95 ft. long and rests on 22 reinforced concrete piles. The piles, extending up to low-water mark only, were driven through the marl until they reached hard rock. On top of these piles the reinforced concrete landing stage was erected, a structure consisting of columns, beams, bracing, and four different decks or landings.

A reinforced concrete dock of magnitude was recently constructed at Port Talbot. Being designed for coaling operations, it is subjected to heavy loads. The designed load was 850 lb. per sq. ft., the test load being 1 390 lb. per sq. ft., covering an area of 720 sq. ft. The outer row of piling consists of two 14-in. square piles encased in a concrete cylinder, 4 in. thick, and 4 ft. 6 in. in diameter. There are six such columns at the face of the dock. Each of the other two rows consists of eight 14-in. square piles. At this port there are also a number of similar structures, the first having been built in 1907.

It is of interest to note that one of the reasons which influenced the engineers in adopting reinforced concrete for the coaling jetties and wharves at Port Talbot Docks was an extensive fire in one of the docks, due to fuel oil which had escaped from one of the vessels. The oil was ignited through carelessness, and caused a very intense blaze over an area of about 250 by 50 ft. though, fortunately, the oil did not spread out under the wooden structures. With the world-wide use of oil as a fuel for the merchant marine, it is well to consider this danger in American wooden dock structures.

Although the Port Talbot Railroad and Dock Company was among the first to adopt reinforced concrete for wharf construction in England, and so was adversely criticized "for using an alleged untried material in such types of work", the results obtained have fully justified this radical departure from what was at that time the prevailing practice the world over.

Scattered all through other English ports are reinforced concrete docks of various sizes.

At Fleetwood, from 35 to 40 miles north of Liverpool, a reinforced concrete fish and coaling dock of considerable magnitude was completed in 1911. The fish shed section is 1 330 ft. long and 26 ft. wide, that is, the reinforced concrete quay part; the filled-in land behind the quay makes the shed sections some 70 ft. wide in all. The coaling section, of similar construction and carrying a coal-loading traveling crane, is 680 ft. long and 26 ft. wide.

At Harwich, 40 miles northeast of the Lower Thames, is the Parkeston Quay, a reinforced concrete dock structure more than 1 000 ft. long and 51 ft. wide.

On the northeast coast of England, at Newcastle-on-Tyne, a reinforced concrete jetty wharf of extensive size lies along the water-front of a large turret shop.

The port authorities at Dundee and Aberdeen, Scotland, are gradually replacing their worn-out wooden dock structure with reinforced concrete.

On the north coast of Scotland, at Ackergill, stands a life-boat slipway, 194 ft. long, built of reinforced concrete, extending out from the rocky shore into the wide open sea. There are several similar structures in other parts of Great Britain.

A reinforced concrete dock was built in the Shetland Islands in 1910. The head of this dock is 80 by 24 ft., and is reached by a concrete approach, 113 ft. long.

In Cork Harbor and other Irish ports reinforced concrete docks of considerable size have been and are being built.

At Portsmouth, Plymouth, and Cardiff are found the first reinforced concrete docks constructed in England, having been built previous to 1906. They are small, and are used mostly for coaling purposes. Quite extensive reinforced concrete docks, constructed during the last year or two, are found at Newport (Mon.), Swancombe, Gravesend, Portencross, South Bank, Ipswich, Newlyn, and numerous other places.

A number of small reinforced concrete docks or pier-heads, other than those just mentioned, exist in England in connection with shipyards, etc. Such docks are found at Dumbarton Shipyard, on the Clyde, and at several similar establishments.

At first the Hennebique system prevailed in all English reinforced concrete dock construction, but, of late, several other systems have been introduced, the most pronounced of these being the Considère spirally armored concrete piling. Though different types of piles are used in English reinforced concrete dock work, there is a most thorough system of diagonal bracing with each type.

In using the Hennebique or other systems of dock construction, where lateral and diagonal braces of reinforced concrete are put in the structure, it appears that trouble has arisen, and might again arise, from the joints in the bracing system. As the foreign docks built on the Hennebique and similar systems seem to have been a success, it does not perhaps cause as much trouble as it did at first, or would appear to cause. In a concrete structure, of whatever design, built in the water, there is always the danger of cracks below the water line, and these cannot be seen and properly attended to.

In a recent address, Mr. Robert Porter, of *The London Times,* stated that "England is one big port". From the vast number of reinforced concrete docks at present in her harbors, it does not seem amiss to say that some day soon the ports of England will be, figuratively speaking, one big reinforced concrete dock, and it will be impossible to enter that country without passing over a structure of this type. In fact, current English technical publications plainly indicate that there

is hardly a port of any prominence along her coast where reinforced concrete construction is not now being carried on extensively, to the extent of five heavy coal tip docks in one harbor alone, due to the great economy of such structures over their old wooden predecessors, in the way of maintenance expenses.

In a recent rebuilding of the old wooden docks or wharves at Plymouth, constructed 30 years ago, the engineers of the Great Western Railroad have stated that concrete construction was adopted because "the cost would be about two-thirds of the cost of rebuilding in timber." These new concrete docks rest on Considère piles, and were designed for a live load of 400 lb. per sq. ft.

In closing this description of English reinforced concrete dock construction, it is well to take note of a few points in favor of this type, as set forth by an English engineer from his experience in that field:

1st. Easy to build;
2d. Indestructibility;
3d. Small cost of annual maintenance; and
4th. Easy to repair.

Spain.

In Spain a number of interesting reinforced concrete dock structures have been built. At Seville, on the Guadalquivir, 50 miles from the sea, are two reinforced concrete ore-loading wharves of some magnitude. At the outer end of each a series of well-based columns supports an ore-loading car-tipping gear. The ore trains run over an extensive concrete viaduct to this tipping gear, and the ore is dumped from the cars by mechanical means into vessels lying at the head of each dock.

A rather striking reinforced concrete dock structure was built several years ago at Aznal Collar mines, also on the Guadalquivir River. It is 532 ft. long and 17½ ft. wide. Each bent consists of two piles capped with a deep concrete beam. As the bent piles are only 10.8 ft. from center to center, the deck system is cantilevered out about 3¼ ft. beyond the supporting piles on each side. The dock carries an electric traveling crane, the track of which runs along the outer edge of the cantilevered decking. The crane has a capacity of 11 tons at a radius of 39½ ft.

France.

In discussing reinforced concrete dock construction as carried out in France, it is of interest to note that the first reinforced concrete pile was made in France by the noted French engineer, M. Hennebique, in 1896, since which time the reinforced concrete pile industry has grown to vast proportions, being used extensively by almost every civilized nation on the globe. From the successful manufacture and use of concrete piling has grown the construction of reinforced concrete docks. Though reinforced concrete construction seems to have been of a negative nature during the close of the 19th century, except in France, its development, as applied to docks, was due to the influence and perseverance of M. Hennebique, and that too in a country other than France; thus reiterating the old saying that "a prophet is not without honor, save in his own country." The first use of reinforced concrete sheet-piling was made at Sable d'Olonne, Bay of Biscay, France. Though these piles were constructed in 1898, as a reinforcement to an old masonry sea wall, they are said to be in as good condition as when first placed.

To record the most important reinforced concrete docks in France: A concrete pile and retaining wall dock was built in 1902 at Nantes, on the Loire, 40 miles from the Bay of Biscay, and in 1912 was reported to be in good condition. At Arcachon, on the Bay of Biscay, not far from Bordeaux, the French engineers have constructed a reinforced concrete recreation pier, several hundred feet long, with some attempts at the artistic, to harmonize with the surrounding landscape. In constructing a quay type of dock at Dives, near the mouth of the Seine, concrete sheet-piles of the Coignet type were used. They are 7½ ft. from center to center, with a concrete slab closing the gap between them. This type consists of a pile with a sort of two-wing arrangement. A cross-section of the complete pile resembles a very flat T-beam. The piles are of 10 by 12-in. section, 26½ ft. long, and the wings are 5 in. thick, and of such width as to make the whole panel pile 4 ft. wide and 18 ft. deep, the pile part thus projecting about 8½ ft. below the panel part. This dock or quay is 190 ft. long. At Fecamp, 20 miles north of the mouth of the Seine, and exposed to the full sweep of the seas, there is a reinforced concrete dock, built in 1911 on the Hennebique system. The depth of water alongside the dock is 28 ft. at high tide. The dock is well braced by diagonal concrete struts, and

has a concrete deck-slab. Further to the eastward, near the Straits of Dover, at Boulogne, there is another reinforced concrete dock, built in 1906 in connection with a quay dock, the reinforced concrete section being 51 ft. wide and 1 050 ft. long.

As France is not a large maritime nation like England, the development of reinforced concrete docks there has not been so extensive as in some of the other European countries.

Holland.

At Ymuiden a reinforced concrete landing stage was constructed in 1903. It consists of two rows of hollow cylinders of reinforced concrete, 8 ft. 2½ in. in diameter and 28 ft. 8 in. in height, filled with sand. The rows of cylinders are 21 ft. 4 in. apart athwart the deck, but 16 ft. 5 in. longitudinally. The deck consists of the usual concrete beam, slab system, and carries two railroad tracks, being designed to support a 45-ton locomotive. The working face of the dock has a wooden fender-pile protection. With the exception of an extension of this landing stage, no other reinforced concrete dock work standing in sea water has been undertaken in Holland; however, extensive reinforced concrete quay walls, etc., have been constructed during recent years at Rotterdam, a fresh-water harbor on the Rhine.

Italy.

Though rather extensive use has been made of mass and reinforced concrete in the development of Italian harbors, etc., the writer has records of only two reinforced concrete docks built in that country. One is at Pozzuoli, near Naples, a regular Hennebique type of pile construction, with a heavy shear leg derrick at its outer end, and the other at Ravenna, on the Adriatic Sea.

Germany.

The overseas shipping of Germany is confined almost entirely to the two ports, Hamburg and Bremerhaven, the former of which is 85 miles from the sea, so that, compared with England, the number of reinforced concrete pile docks standing in sea water in Germany is exceedingly small. Although the port of Hamburg is equipped with the most modern shipping facilities, most of the docks are of the heavy masonry quay type, and not the long piers so common in America. This

is due to the narrowness of the Elbe as compared with the broad expanse of water-front of other ports. Although the writer's file is lacking in records of reinforced concrete pile or similar docks as having been constructed in Germany, no doubt they exist and are of considerable magnitude. It is hoped that some of the members of the Society may be able to furnish full information concerning them.

As to reinforced concrete dock construction in Sweden, Norway, Denmark, and Russia, the records are extremely meager, perhaps because there are no docks of this type in these countries. Russian engineers, however, have built several concrete docks in the Baltic Sea, there being one at Touapse, a jetty built of reinforced caissons.

Although little, if any, information seems to be available covering the harbor work, etc., of the eastern part of the Mediterranean, in Grecian, Turkish, and Asia Minor ports, it would be plausible to consider that the concrete industry is well known in that part of the world, and that it has been used to considerable extent in eastern Mediterranean ports as well as in the Black Sea, in the construction of not only graving or dry docks, but also reinforced concrete docks and quays.

Africa.

At Alexandria, Egypt, there is a reinforced concrete lighthouse, in 30 ft. of water, exposed to violent storms and heavy seas. Its base is a truncated pyramid 45 ft. high. The slender main tower extends some 33 ft. above the base, its top being about 48 ft. above sea level. At one time this lighthouse was rammed by a 3 800-ton steamer, but it does not seem to have been injured, except that, since that time, it has leaned 4° from the vertical.

At Senegal, on the French West Coast, a reinforced concrete pile dock and jetty was built in 1912. The dock is 242 m. (794 ft.) long, of varying widths, covering an area of 2 500 sq. m. (26 900 sq. ft.). The jetty head is 60 by 10.3 m. (196.8 by 33.8 ft.), with an approach 140 m. (461 ft.) long and 2.65 m. (8.7 ft.) wide, giving the jetty a total area of 1 300 sq. m. (13 990 sq. ft.).

At Cape Town, a long reinforced concrete pier was finished recently. The concrete pile part is 800 ft. long and 45 ft. wide, except for the outer 75 ft., where it is 65 ft. wide. The piles are 14 in. square, 200 being used in the structure. They were driven in groups of four, 13 ft. and 10 ft. from center to center, with a longitudinal clearance of

27 ft. and an across-dock clearance of 18 ft. between each group of four, thus enabling small boats to pass under the dock as well as down the center aisle, as it were. The outer 75 ft. is well braced, the piles being close to one another.

Australia.

Australia and New Zealand are far distant, and thus perhaps few data reach the American engineer as to their developments in concrete. From what is known of several famous concrete structures in that part of the world, these countries are well in the fore as respects reinforced concrete. In fact, at Auckland, New Zealand, there are from four to six reinforced concrete docks, two of which, at least, are of extensive size, *viz.*, a railroad dock, 1 500 ft. long and 240 ft. wide, and Queen's Dock, 1 200 ft. long and 280 ft. wide.

It is of interest, in passing, to take note of a reinforced concrete lighthouse standing in the Straits of Malacca, a two-story closed-in structure, supported by seventeen reinforced concrete piles. On top of the main structure there is an open part consisting of reinforced concrete columns and braces supporting the light and its house.

India.

At Madras three reinforced concrete docks, of the bulkhead type, were constructed in 1911. They consist of a series of concrete piles driven uniformly 8 or 10 ft. apart, depending on the type of construction used. Back of these piles were placed reinforced concrete slabs, in the same way that wooden planks are placed back of and secured to wooden timbers where bulkheads are built to retain earth.

In one type, where the water is only 6 ft. deep and the dock has a freeboard of 10 ft., 18-in. piles, 10 ft. apart, with a penetration of 18 ft., were used. The thickness of the two lower concrete slabs is 12 in., and that of the top slab 15 in.

In another type, where the water is 9 ft. deep and the dock has 6 ft. of freeboard, 15-in. square piles, 8 ft. apart, were used, the concrete slabs being 6 in. thick. The tops of the piles are connected by reinforced jack-arches. In the third type, the piles are 15 in. square, 10 ft. apart, and are tied back to anchors 30 ft. behind the face of the dock. The water is only 4 ft. deep at the face of the dock. Each of these three walls is about 1 500 ft. long, their total length being 4 700 ft.

China.

What is said to have been the boldest undertaking in the construction of a reinforced concrete dock is found at Shanghai on the Whang Poo River, 50 miles from the open sea. The great difficulty encountered in the building of this structure and its accompanying warehouses was the excessive depth of the overlying river silt. The design of the structure itself presents nothing especially new, in comparison with other dock structures, as built in England. The fact that nothing but mud of various degrees of solidity, with layers of impure sand at irregular depths and a thin layer of gravel at a depth of 300 ft., covered the location of the proposed structure for a tested depth of 400 ft., gave a most unsatisfactory soil on which to construct a dock. At a depth of about 400 ft. there was a bed of gravel. The top layer of 25 ft., into which the concrete piles were driven, consisted of a solid crust of stiff sandy clay crossed with old creek beds filled with soft mud, rendering the surface somewhat irregular. The load on the piles had to be carried by the skin friction of this upper layer of supposedly firm ground. From a careful study of the situation, and from deduction obtained from old timber docks built in the same soil, a factor of 500 lb. per superficial foot of pile was fixed as the maximum carrying capacity of the piles, exclusive of their own weight.

The dock structure is 1 160 ft. long and 174 ft. wide, the rear 160 ft. being covered with two steel-frame sheds arranged in such a way as to leave an open space of about 250 ft. between them as an entrance way to the main office building. The decking of the dock was designed for a load of 500 lb. per sq. ft., with a deck load of 350 lb. per sq. ft. for the general structure. In general, the structure consists of a 5-in. deck-slab, deck-beam, and girder system of concrete construction, carried by 15 by 15-in. concrete piers, 15 ft. from center to center, each pier being supported by four 14-in. square concrete piles. At the face of the dock, where there is a large traveling crane, the columns are 18 in. square, and are carried by a group of six 14-in. square piles. The designed load per pier was 45 tons, or 11 tons per pile. The structure has the usual horizontal, cross, longitudinal, and diagonal bracing ties.

Due to the extraordinary conditions under which this dock was built, it is of interest to note a few facts as regards the pile-driving and the final loads carried. The piles were driven with a 3½-ton steam

hammer, the weight of the cylinder, which acted as a dead load on the pile during driving, being 1½ tons. The piston stroke was from 6 to 8 in. The piles had a penetration of about 25 ft., the final set being 1 in. under the last blow. It is stated by the engineer of this structure that "no anxiety was felt as to the future stability of the majority of the piles" under such an excessive set, as it was considered that their supporting power would be increased after the skin of water between the mud and pile had escaped and true skin friction was established. The resistance to driving increased very little after a 15-ft. peneration had been reached, and sometimes was less for a penetration of 25 ft. and above. On one occasion the piles sank so rapidly under the imposed dead weight of the hammer and cylinder that their descent had to be checked by holding up the dead weight of the hammer at the desired elevation. After 4 weeks, this group of four piles was tested with a load of 45 tons, and there was a 1¾-in. setting in 2 weeks, when the settlement ceased. The piles were lengthened 8 ft., and then driven deeper. Though it took 80 blows to re-start the piles, they were finally driven with the same ease as at first. A subsequent test of 45 tons gave a settlement of 1½ in. No additional attempts were made to lengthen any of the piles. When completed, the dock was loaded to more than that for which it was designed (1 000 lb. per sq. ft., by mistake), with no sign of any settlement.

In general, the piles were from 6 to 8 weeks old before they were driven. The range of temperature was about 150°, which appeared to cause hair cracks in winter along the lines where work was stopped in the deck-slab. One of the leading dock engineers of England has remarked that this dock illustrates the great advantages of a concrete decking in its ability to distribute the superimposed loads and thereby assist the piles to carry a heavy burden. The larger part of this structure was moulded on land and put in place on the unit system, as it were, especially those parts below water, except the caps. In connection with this dock, there was also built a quay wall about 495 ft. long and 21 ft. wide, on one side of the property, for the berthing of lighters.

Japan.

The writer has no record of any reinforced concrete pile docks having been built in the development of Japanese harbors, yet he has no doubt that they exist. For 20 years, at least, the Japanese have

been using concrete in breakwaters to an enormous extent, and it is only a part of wisdom to consider that they have used it in other reinforced concrete structures in sea water. Recently, extensive developments have been made in the harbor of Kobe, where four concrete piers have been built, and of a most unique type of construction, *viz.*, reinforced concrete caissons built in the dry on a pile structure on the shores of the harbor, then lifted off this foundation by a specially designed depositing dock, carried into deep water, and put afloat by sinking the depositing dock and withdrawing it from beneath the concrete caissons. The caissons were then towed to and sunk in their proper places in the quay wall. Each caisson, 119 ft. long, from 35½ to 41½ ft. high, and from 24 to 30 ft. wide, has twenty open-top cells, and weighs from 1 900 to 2 400 tons.

Philippines.

It may not be exact to regard the Philippines as a foreign country, so far as concerns engineering work now being done, yet, in this paper, it has been classed as such, and note is here made of the reinforced concrete docks in those waters. The first dock of this type was built in Cavite at Manila in 1902, on what is known as the "Cushing System of Piers", *viz.*, cast-iron cylinders surrounding wooden piling and filled with concrete, the deck system consisting of steel girders and beams covered with a concrete slab. The dock is 408 ft. long, 75 ft. wide, and contains thirty-six cylinder piers 6 ft. in diameter. The structure was built by the United States Navy Department in connection with a coaling station.

Two docks of a similar design were built in 1906 by the Navy Department at Manila. One of these is 650 ft. long and 110 ft. wide; the other is 600 ft. long and 70 ft. wide. There are nearly 300 cylindrical piers in the two structures.

The Navy Department has also constructed two reinforced concrete docks at the Naval Station, Olongapo. One of these is 332 ft. long and 45 ft. wide; the other is 300 ft. long and 60 ft. wide. The piers or columns of both consist of hollow concrete cylinders, 2½ ft. in diameter, filled with concrete, and 12 ft. apart longitudinally and 18 ft. transversely. The decking consists of I-beams, encased in concrete, and these support a suitable concrete slab. There are light diagonal ties at each bent, running from the base of each outer column to the

top of each inshore column, but there does not seem to be any other lateral bracing.

Another reinforced concrete dock has been finished recently in the Philippines, at Iloilo, about 330 miles south of Manila. It runs along the face of an old sea wall, and consists of a series of transverse girders, 10 ft. from center to center, with a 12-in. concrete deck-slab. The outer end of each girder rests on a hollow cylindrical column, and the land ends are seated on a concrete pedestal built up from the natural slope of the bank. At the face of the wall, arches are inserted between the girders in order to provide additional strength to resist lateral stresses, as well as to give to the dock an artistic finish.

South America.

In view of the extensive harbor developments and the magnificent systems of docks, etc., in some of the ports of the leading South American countries, it is to be regretted that more general information concerning them is not readily available for the engineers of the United States. These engineering undertakings and port developments stand out as the most prominent of such world undertakings, especially at Buenos Aires, which city is conceded to have the finest shipping and port facilities in the world, with Montevideo a close second. It has been stated that almost all South American docks of any magnitude are being constructed of concrete, though the percentage of reinforced concrete structures among them is unknown to the writer.

In the foregoing the writer has attempted to outline the most important concrete docks, etc., constructed up to date in European and other countries, without any attempt to go into the technical side of their design, though some of them are more in the nature of semi-concrete and concrete steel docks than full concrete structures. The list is far from complete, but gives a general idea of the extensive use of reinforced concrete in dock construction in foreign ports.

Part II.—Reinforced Concrete Docks in the United States.

In recording what has been accomplished in the construction of reinforced concrete docks in North America, including Porto Rico, Cuba, the Canal Zone, and Canada, such undertakings are so few and of such recent date, compared with those in European countries,

that the art of building reinforced concrete docks there may be said to be hardly beyond its infancy, especially as regards out-and-out docks or piers, as the American usually understands the word, *viz.,* long structures running out from the shore in such a way that vessels can lie on each side. Unfortunately, it will be necessary to make note of some failures among North American concrete docks.

Atlantic Coast.

Boston.—The first concrete dock built in Boston Harbor has perhaps caused more discussion as to the feasibility of using concrete in sea water than any other American structure of this type, and, therefore, is far-famed in itself. This dock, or pier, was built at the Charleston Navy Yard about 14 years ago. The first section, consisting of a long, straight wall, was built in 1899-1900, without any resort to a coffer-dam. The other two sections, consisting of plain reinforced arches, 20 ft. wide, with spandrel walls, were constructed in 1901, the space between the two walls being filled with earth and stone. The first section was built of 1:2:3 concrete throughout, and it was all placed in the wet, with an open-top bucket. The second and third sections were of different mixtures, the main body being of 1:3:6 concrete, the outer 2 ft. consisting of a 1:2:4 mixture, and the whole exposed surface was faced with 3 in. of 1:1 mortar. In these two sections the concrete was placed very dry, as was the practise at that time.

During 1913 one of the Boston public docks, Pier No. 5, was partly rebuilt in concrete. This dock consisted of a timber platform deck, 50 ft. wide and 1 150 ft. long, on each side of a solid earth fill. The new wooden piles needed were driven, and all the piles, new and old, were cut off at mean high water and capped with a deep reinforced concrete beam running athwart the piles, in the way of curtain-walls. These curtain-walls support two longitudinal reinforced concrete beams, the third or inner beam resting on the earth retaining wall. There are also two additional concrete beams under the track along the outer face of the dock, with special supporting piles. On top of the curtain-wall and longitudinal girders is laid a reinforced concrete floor-slab with a 2-in. bitulithic top. Wooden longitudinal tie members are run from top of pile to top of pile under the concrete curtain-walls, the whole timber structure being well braced with piles and wooden ties. To provide against accident from disintegration

of the lower part of the curtain-walls, due to frost action or other cause, cast-iron columns, 30 in. long, are attached to the top of the piles and made integral with the curtain-walls, thus guarding against weakness in the concrete, rather than in the piles, as done in other harbors.

New York Harbor and New Jersey Coast.—A reinforced concrete dock of an experimental nature was constructed at Ellis Island, New York Harbor, in 1911. It is a rather small structure, 30 ft. wide and 50 ft. long, resting on thirty-six driven concrete piles 18 in. square. The piles were made of different mixtures, for experimental purposes, and various kinds of water-proofing were used in order to determine their efficiency under the same conditions. This was the first complete concrete pile and deck dock built in New York Harbor.

During the past seven years a semi-concrete type of dock has been under development in New York Harbor, *viz.*, wooden piling, wooden caps, and concrete decking. In one dock, on the New Jersey side of the Hudson, the caps on top of the piles are also of concrete. In the final type, as worked out by the Department of Docks and Ferries,* the concrete slab rests directly on wooden caps secured to the tops of the wooden piles, a genuine flat slab between bents, the entire timber floor system being wholly eliminated. At present some 25 or 30 semi-concrete docks have been built on this system in New York Harbor. It has been stated authoritatively that they have proved a great success.

In building two semi-concrete docks at the Brooklyn Navy Yard a few years ago, the objectionable features of docks of the foregoing type—*viz.*, part of the wooden pilings and bracing exposed to wet and dry conditions between low water and the decking, and the wooden cap as an additional temporary item helping to support a permanent structure—were eliminated. In the two Navy Yard docks the wooden piles were cut off a little above low water and capped with a wooden grillage. Pre-moulded concrete columns, mixed with water-proofing compounds, were set on and dovetailed into the caps, and a concrete girder-beam and deck-slab system was worked over the tops of the columns. The wearing surface consists of a creosoted wooden block pavement. Down each side of the dock there is a standard-gauge railroad.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 503.

A small concrete dock was constructed at Glen Cove as a yacht-landing, in the winter of 1909-10. It consists of eight reinforced concrete rock-filled caissons, supporting an overhead footbridge, the total length of the pier being about 330 ft.

Long Branch.—At Long Branch a Hennebique type of concrete pile dock was constructed in 1911, running some 848 ft. out into the Atlantic Ocean, as a boat landing and recreation pier. At present the pier is only 75 ft. wide, except for an 80-ft. length at its outer end, where its width is 150 ft., the intention being to make the whole pier of that width at some future time. The deck is 22 ft. above low water. The piles are 16 ft. from center to center longitudinally, but 20 ft. from center to center across the pier, except the outer two rows, which are 15 ft. from center to center. Most of the piles are of hollow cross-section, 22 in. external diameter, 13 in. internal diameter; the penetration was about 22 ft. To provide sufficient impermeability, the shells of the piles were made of $1:1\frac{1}{2}:3$ concrete, the fill being of a weaker mixture. Apparently, no cross-bracing system was used, the outer end of the pier being stiffened laterally by inclined bracing piles at regular intervals.

Atlantic City.—At Atlantic City the famous steel pier was widened and protected in 1906 by the use of concrete. The original pier was founded on steel-pipe piles resting on cast-iron disks, a type of construction quite common during the last part of the 19th century. The pier extends out into the Atlantic Ocean a total length of 1 600 ft. In rebuilding the pier all the original metal work was encased in concrete and the pier was widened on both sides, 12-in. and 25-in. reinforced concrete piles, with enlarged footings, being used. The smaller piles were pre-moulded vertically, on a small platform, each pile at its final location. After hardening sufficiently they were lifted off their platforms and jetted into place through from 8 to 14 ft. of sand. The larger piles were given a penetration of 16 ft. The lower 12 ft. of these piles were pre-moulded on a platform, a water-tight iron casing was secured to the upper end, and the whole was jetted into place. The dry caisson was then filled with concrete up to the proper level, the maximum total length of the 25-in. piles being 52 ft. In protecting the original piles, concrete shells were cast around them, with sufficient interior clearance, and the space was afterward filled with grout.

After the concrete work of this structure had been in the sea water for 6 months, the piles became coated with a sort of gelatinous matter which seemed to act as a most excellent protective coating against any deterioration. The same peculiar action has also been noticed in California.

Although not exactly a dock, it is of interest to note the concrete pile Boardwalk at Atlantic City. Not only is it necessary to guard against dry and wet conditions at such resorts, but the fine sands act like a sand-blast when driven like snow before the wind. In 1908 part of the old wooden structure was rebuilt with 16-in. concrete piles, supporting a concrete cap, and that in turn carried the wooden decking.

Baltimore.—It is perhaps at Baltimore that the most extensive reinforced concrete docks on the Atlantic seaboard have been built. Although the water in Baltimore Harbor may not have the same density of salt as in ports nearer the sea, these docks, thus far, have shown no sign of deterioration, though at times subject to frost action. Three of these piers are of a back-filled concrete bulkhead type, and are not docks resting on piles. Pier No. 4 is 978 ft. long and 220 ft. wide; Pier No. 5 is 1 245 ft. long and 200 ft. wide at the shore end, but 243 ft. wide at the water end; Pier No. 6 is 1 456 ft. long and 93 ft. wide at the shore end, but 212 ft. wide at the water end; all were built in 1908.

In general, these three docks consist of a series of oval-shaped concrete cylinders 25 ft. apart along the face of the docks, and sunk to about 25 ft. below low water. Along the face of the cylinders, and just above high water, there is a concrete-encased iron girder, tied back to a deadman some 28 ft. in the rear of each cylinder. A row of concrete sheet-piling was driven back of the girders to form a vertical retaining wall, the upper ends of the sheet-piling bearing against the girder, and the lower ends being driven into the muddy bottom. A horizontal box-girder encased in concrete runs along the upper face of the dock, supporting the outer edge of the concrete curb slab, on which are laid the paving blocks. The cylinders are tied together in certain cases by ties extending entirely across the docks. The face of each dock is protected by wooden fender-piles, 8 ft. apart. Another concrete dock has been completed recently in Baltimore by the Harbor Commission, the details of which are lacking. In the

same harbor is found a concrete bulkhead dock, built for a private corporation—a reinforced concrete sheet-piling structure capped with a concrete girder tied back to deadmen by reinforced concrete ties.

At Sparrows Point, near Baltimore, a reinforced concrete ore dock, 600 ft. long, was built in 1911. It consists of two parallel concrete walls, about 46 ft. apart, *viz.*, (1) a sheet-pile bulkhead on the water-front capped by heavy concrete girders with a cantilevered shelf, as it were, on the outer face, running the full length of the bulkhead; (2) a heavy retaining wall in the rear, the two walls being tied together by reinforced concrete ties about 30 ft. apart. The back wall, resting on wooden piles, not only acts as a deadman for the outer wall, but affords a means for carrying one track of the large, heavy, ore unloading crane that straddles the filled-in space between the two walls, the front track of the crane running along the outer wall. The dock face is protected by a substantial system of fender-piles and wales with heavy helical car-springs at each buttress of the face wall.

Norfolk.—In constructing the Virginian Railway Coaling Terminal, at Sewells Point, in 1907, it was not practical to carry the massive steel superstructure on creosoted piles. In place thereof groups of wooden piles were driven and cut off 1 ft. below the mud line. On top of these piles were built monolithic concrete piers, of pyramidal shape, to 4 ft. above high water. All the concrete work was done in the dry, inside a coffer-dam. These piers are reported to be in as good condition as when first built.

Brunswick and Charleston.—Perhaps the most extensive development of concrete dock construction, combining concrete piles with wooden decking, is found at Brunswick, Ga., and at the U. S. Navy Yard, Charleston, S. C., built in 1906.

The Brunswick terminal consists of two piers, 500 and 900 ft. in length, respectively, and each is 140 ft. wide; there is also a coaling pier about 300 ft. long. The 16-in. bearing piles, of pre-moulded concrete, are 12 ft. from center to center each way. They are from 30 to 51 ft. in length, with the lower 10 ft. tapering to 8 in. The piles have a penetration of 40 ft. Their upper ends are corbeled out to support the double 8 by 16-in. wooden caps. The decking consists of 6 by 14-in. stringers and a 3-in. flooring. Each bent is well braced with creosoted wooden cross-bracing.

The Charleston Navy Yard dock is 60 ft. wide and 520 ft. long, and of the same type of construction as the Brunswick structures. The piles, 10 ft. from center to center each way, are 18 in. square, instead of 16 in. square, and have an 8-ft. taper to 12 in. square at their lower ends, thus giving a heavier structure than those at Brunswick. The test load on the Charleston dock was 30 tons per pile for 48 hours, though the specification required only 20 tons, or 400 lb. per sq. ft.

The outer row of piles in these docks consisted of three creosoted yellow pine sticks, two of which were driven on a batter; all were bolted together to afford sufficient protection to the dock in the form of a fender-pile system.

During the building of the Brunswick dock it was rammed by a large steamer. Although a number of the pine piles were broken, it has been stated that the concrete piles withstood the shock successfully.

Savannah.—A rather unique type of concrete dock was built at Savannah, 17 miles from the sea, in 1913. The design seems to contain many of the excellent features of the Ambursen dam. This dock consists of a series of pile bents athwart the dock supporting reinforced concrete brackets of triangular shape, the brackets in turn supporting a concrete deck-slab sloping down and toward the rear of the structure. This deck was afterward back-filled and finished off with a suitable working face. The dock forms a water-front structure, and is protected by wooden fender-piles.

Jacksonville.—In the construction of a semi-concrete dock at Jacksonville, the Braxten concrete pile was used, with very satisfactory results. In another case the Ripley concrete-encased wooden pile was adopted.

Key West.—A reinforced concrete quay wall dock, 1 589 ft. long, was completed at the U. S. Navy Yard at Key West in 1912. The main wall consists of a series of pre-moulded concrete pile bents capped by a concrete girder and a deck-slab 40 ft. wide on top. From the inner edge of the deck-slab a sloping concrete apron runs down to the top of a row of sheet-piles which forms a retaining wall for the reclaimed land. The piles are from 16½ to 20 in. square, and vary from 25 to 60 ft. in length. The bents are 10 ft. apart, with the same spacing for the piles, each bent having six piles. The face of the dock is protected by a system of creosoted fender-piles placed midway between each bent.

Port Arthur.—A reinforced concrete pile-bent dock, 1 050 ft. long and 25 ft. wide, was constructed at Port Arthur, Tex., in 1911-12. In general, the piles are 16 in. square, 44 ft. long, and 5½ ft. from center to center. The pile bents, of five piles each, are about 23 ft. apart, and are capped with a reinforced concrete girder. Five concrete beams, running from bent to bent, and a 4½-in. concrete slab, form the deck structure. The dock is tied back to the concrete trestle built for carrying the railroad tracks in the rear of the dock. No provision is made for any spring or other device to take up the impact forces on the fender system, as it is believed that the wooden fender-piles will afford sufficient elasticity to prevent any injury to the dock from this source.

Cuba.—Two reinforced concrete docks, 620 and 670 ft. in length, respectively, and 160 ft. wide, were built in 1911-12 at Havana, the depth of the water varying from 12 to 40 ft. Each consists of a concrete floor-slab resting directly on concrete caps placed on top of clusters of from four to eighteen reinforced concrete piles, the clusters being about 23 ft. from center to center in each direction. The concrete piles, 18 and 20 in. square, were designed for a load of 32 tons each. The design of the floor slab would indicate a cantilever effect longitudinally between each row of longitudinal piling.

One of the railroad companies of Cuba, also, has built a reinforced concrete dock at Havana, for coaling purposes. The structure, which is subjected to very heavy loading, rests on Chenoweth concrete piling, and was but recently finished.

Haiti.—In constructing a reinforced concrete dock at Port au Prince, during 1913, the Ripley type of concrete wrapped wooden pile was adopted. This dock has a total length of 2 326 ft., varying in width from 24 to 60 ft. The piles are 10 ft. from center to center, longitudinally and transversely, and are capped by heavy concrete girders of rectangular section for the inshore end of the dock, otherwise by arched girders. The deck system consists of a series of reinforced concrete beams supporting the concrete deck-slab, built with a crown, in order to shed water. The dock is protected by a creosoted fender-pile system.

Panama and Canal Zone.—The United Fruit Company in 1909 built a combined reinforced concrete and wooden pile dock at Bocas del Toro, for the docking of fairly large steamers. The wooden piling is surrounded by a 4-in. concrete shell up to about 1 ft. above the

high-water line. The piles are extended up to the deck as reinforced columns, with a concrete beam and deck-slab system. Up to the present time the dock is said to have given good results.

In the Canal Zone the U. S. Army Engineers have constructed a reinforced concrete dock, 706 ft. long and 55 ft. wide, for unloading timber. There are fifty-five concrete piers or columns, 8 ft. in diameter and about 80 ft. long arranged in two rows, 35 ft. from center to center across the dock, and 30 ft. from center to center longitudinally, and built in the form of hollow reinforced concrete sectional cylinders. After these cylinders had been sunk to bed-rock, they were filled with concrete, being reinforced vertically with eight rails. On top of the columns there is a concrete girder, deck-beam, slab system. The girders are about 5½ ft. deep, the beams about 4½ ft. deep, and the slab 6 in. thick. The railroad track runs over one row of columns. The floor system is designed for a load of 400 lb. per sq. ft., with a concentrated load of 105 tons over the track beams. The depth of water for a mean sea-level tide is 40 ft., the total fluctuation in the tide being 20 ft.

Two other concrete docks of extensive size are now in course of construction in the Canal Zone, with still more to follow.

Pacific Coast.

On the long stretch of our Pacific Coast, perhaps is found the greatest development of reinforced concrete dock construction in the United States. This section is making vast harbor improvements in anticipation of the opening of the Panama Canal.

San Diego.—At this most southern port on the California Coast an extensive reinforced concrete dock is now under construction. It consists of two parts, *viz.*, the dock itself, 800 ft. long and 130 ft. wide, and a quay wall or bulkhead, 2 675 ft. long and 25 ft. wide. Wooden piles are driven into the soil and cut off "at any point between mean low water and 18 ft. below city datum." Each of the 42-in. concrete columns encases one wooden pile and supports a system of structural deck-beams, a concrete slab covering the whole. The columns are 15 and 13 ft. 4 in. from center to center. The entire structure is protected by a wooden fender-pile system having the so-called San Francisco type of steel spring shock-absorbers.

San Pedro.—In connection with extensive port developments at San Pedro, a semi-reinforced concrete dock was recently completed in the outer harbor. It consists of pre-moulded concrete piles, 10 ft. from center to center in each bent, the bents being 16 ft. apart. The tops of these piles are corbeled out to support two 10 by 16-in. wooden caps, which in turn support the wooden floor joist and wooden decking. The piles are tied together with a wooden cross-bracing system above mean high tide. The structure is also stiffened against lateral blows on its face by inclined bracing piles. The wooden pile fender system has a car-spring to assist in taking up lateral forces. The dock is of the quay type, 48 ft. 6 in. in width, the total pier head frontage being 12 000 lin. ft. A railroad runs parallel to the inner edge on the inshore fill.

Redondo.—It is of interest to take note of the ocean pier at Redondo, Cal., though it is not a dock. It extends some 637 ft. out into the Pacific Ocean, and supports the intake pipe (for cooling purposes) of a power station. The pier consists of concrete pile bents, 20 ft. apart, each bent having four piles. As considerable surf runs at times under this pier, the outer bents have an extra outside pile driven with a batter of 2 in. per ft. The piles consist of a thin steel shell, 18 in. in diameter, closed at the lower end. After the steel cylinder had been driven to the proper depth of penetration, the reinforcement was inserted and the cylinder was filled with concrete. The piles of each bent have a structural steel cap encased in concrete, with a system of diagonal bracing in a horizontal plane connecting the tops; there is also a longitudinal system of ties at the tops of the piles, above the reach of the water. This structure, as a whole, is said to possess considerable elasticity.

Long Beach, Cal.—In 1907 a concrete pile pier was built at Long Beach, extending some 1 300 ft. out into the ocean. The head of the pier is 100 ft. long and 300 ft. wide, the approach being 1 299 ft. long and 32 ft. wide. The deck is 30 ft. above mean low water, so as to be kept clear of the 24-ft. waves which at times roll in from the ocean. The piles and columns are 4½ ft. in diameter, and are arranged in bents. Under the head of the pier the bents and piles are 16 ft. from center to center. The approach bents are 20 ft. apart, with two columns each. The columns are sunk from 10 to 18 ft. into the sand, in order to be absolutely safe, as it is said that at times the undertow

digs out the sand to a depth of 13 ft. The pile caps consist of steel I-beams, on top of which rest wooden stringers carrying a wooden decking.

Santa Monica.—At Santa Monica, an ocean pier, built in 1908-09, extends 1 600 ft. out into the ocean. It was built on driven concrete piling, ranging from 14 to 22 in. in diameter, in lengths up to 75 ft., with from 16 to 20 ft. penetration. Although the pier was built primarily to support the outfall sewer carrying the sewage effluent to a point far seaward, it is also used for recreation purposes. The pier is about 35 ft. wide at the deck line, with three platform spaces of 43 by 89 ft. at intermediate points and at the end. The bents are 20 ft. from center to center and consist of three piles, the piles being 13 ft. 6 in. from center to center. Each bent has a concrete cap on which rest the wooden joists covered with 2-in. planking, and the latter is covered with a 3-in. wire-mesh concrete slab, having the proper pitch to carry off the water. The bents are tied together by three longitudinal reinforced concrete tie-beams running from top of pile to top of pile. The piles are bulb-pointed.

San Francisco Bay.—Although the dock engineers of New York City have developed a type of semi-reinforced concrete dock, *viz.*, a wooden pile structure supporting a concrete slab, especially adaptable to local conditions, the dock engineers of San Francisco have developed a type of full reinforced concrete dock based on wooden sub-piling, concrete column piers, steel or concrete deck-beams, and concrete floor slab, the concrete encasing the steel beams and the floor being made monolithic, with details varying to suit special conditions. Although the type as worked out presents no difficulty in the way of construction, outside of building the main columns, that part of the work has been done successfully, but with considerable difficulty. The mud line at the bottom of the bay is said to be approximately level, yet, at the outer ends of some of the piers there is a depth of only about 18 in. of mud over the rock; at the shore end, however, there is a depth of 35 ft. of mud. Piles can be driven to a rock bearing in some places, but it is impossible to use wooden piles throughout. Along a portion of the water-front, where it is not possible to reach the rock, there is a hard soil capable of bearing from 4 to 6 tons per sq. ft., thus doing away with the necessity of any sub-piling.

The method used in building the column piers is to sink a hollow

steel caisson, of such length that it will not be overtopped by the water, dredge out the interior to the desired depth, and build the reinforced column in the dry. In some cases the columns rest on solid rock, in others, wooden piles have to be driven inside the cylinders to obtain the necessary bearing support. The size of the columns varies according to conditions. In two docks built in 1910, 140 ft. wide and 780 ft. long, where the mud covering the rock was less than 50 ft., the columns were seated directly on the rock. Where the mud is more than 50 ft. deep the columns rest on five 15-in. wooden piles driven to refusal, the piles being cut off 35 ft. below the water line and encased by the concrete columns to that height. The columns are 6 ft. in diameter to a height of 7 ft., and then 3½ ft. to the top.

In laying out a vast dock improvement proposition at Fort Mason, San Francisco, the Government has planned for the immediate construction of three docks of the usual San Francisco type, each to be 500 ft. long, two 81 ft. wide, the third 118 ft. wide. The concrete columns are to be supported by groups of seven wooden piles driven in a circle 6½ ft. in diameter, and 18½ ft. from center to center each way. The piers are to be 8 ft. in diameter up to 12 ft. above the dredge line, and are then to be reduced to a diameter of 4 ft. for the remainder of their length. The wooden piles will extend some 11 ft. up into the concrete columns, the bottom of the concrete being well below the mud line. In building the first of these docks, an attempt was made to construct the column forms of 4-in. staves, sufficiently reinforced with bands, and sink them into position by driving. The method did not prove a success, and resort was made to the steel cylinder caisson method, as described previously.

Up to 1911 there were only four modern reinforced concrete column docks under the control of the San Francisco port authorities. Since that time they have added largely to the number by replacing some of the older wooden pile docks with reinforced concrete structures. The first addition was Pier No. 17, 800 ft. long and 126 ft. wide, with suitable railroad track accommodations. It consists of wooden piles protected by concrete shells, the deck-beams being of structural steel encased in concrete, and the stringers and decking are of timber—a sort of semi-concrete pile semi-concrete dock.

The next docks reconstructed were Piers Nos. 26, 28, 30, and 32, all of the same type, having reinforced concrete columns resting on

the hard bottom, without any piling, with a complete system of reinforced concrete deck-beams, girders, and slabs. These docks are equipped with up-to-date cargo-handling machinery.

The next addition was Pier No. 39, 150 ft. wide, this being in process of construction at the present time. The concrete columns rest on groups of from 4 to 10 wooden piles, the entire deck system being of reinforced concrete.

In another type of construction at San Francisco the wooden piles are wrapped with wire fabric, or otherwise, and a concrete shell is placed around them after the piles have been driven to place. This method, apparently, has proved successful, though it must be carried on in such a way that the concrete can be poured, set, and hardened in the dry, and not in sea water, if permanent results are to be obtained.

Recently, the City of Oakland, Cal., built a genuine reinforced concrete pile dock, 295 ft. long and 124 ft. wide, standing in 30 ft. of water. The piles are of pre-moulded concrete, 16 in. in diameter, octagonal, and of a $1:1\frac{1}{2}:3$ mixture for a distance of 5 ft. from the top, the remainder of the pile being of a $1:2:4$ mixture. The bents and piles are approximately 10 ft. apart. Each row of piles has a concrete cap or girder running athwart the dock, the deck-beams and deck-slab being also of concrete. For lateral stiffness, 12-in. concrete curtain-walls were built at three points in the dock, for about one-third of its width, between the piles in three bents.

Portland.—Being 112 miles from the ocean, on a fresh-water river, it is possible to use wooden piling at Portland, in the construction of docks. A massive concrete dock terminal, now being built by the city, consists of four concrete warehouses along the water-front. The dock part of this project as designed consists of a reinforced concrete platform, 1 030 ft. long and 100 ft. wide, 32 ft. above low water, resting on wooden piles driven in groups and cut off at about mean low water. Resting on each group of wooden piles, 20 ft. from center to center, are the reinforced columns supporting the upper platform, composed of steel I-beams encased in concrete, and a concrete floor-slab. For a length of 300 ft., a low-level deck, 14 ft. below the main deck, is provided. As the Columbia River is subjected to high- and low-water stages, due to floods, it was necessary to provide this lower platform for use by river steamers during low-water periods. The rise

and fall of the Columbia River attains a maximum of about 28 ft., though 18 ft. is about the average, all based on mean low-water level. Thus the lower platform will seldom be under water.

Puget Sound.—Though there are several large shipping ports on Puget Sound, up to the present time, no reinforced concrete dock construction of a commercial nature has been undertaken in these waters. As lumber is so plentiful in that part of the country, it is only natural that such a section should be one of the last shipping centers to take up the building of reinforced concrete docks; but as the destructive teredo is very active there, the engineers of the Northwest are beginning to seek a more stable type of construction than creosoted wooden pile docks, especially the United States and Canadian Governments, and some of the railroads, because "in Government [and publicly owned] docks, a small saving in first cost is of minor importance, but weakness and frequent need of repairs are well nigh intolerable. On the other hand, in private ownership of docks a saving in first cost is usually of serious importance, while the cost of maintenance, repairs, etc., is met by earnings of the dock and is less felt", unless they become so excessive as to make a concrete proposition more economical in the long run.

At the U. S. Navy Yard in Bremerton, Wash., a reinforced concrete dock, consisting of concrete columns, steel and concrete beams, and a concrete slab, was completed in 1912. It is 402 ft. long and 60 ft. wide. The columns are 3 ft. in diameter, with a flared-out footing to a diameter of 6 ft., 16 ft. from center to center, each way, there being four columns to each bent. The caps and girders over the tops of the columns are of I-beams encased in concrete. A concrete beam is run midway longitudinally between each of the four steel beams. The I-beam caps are cantilevered out 6 ft. on each side of the dock. The columns were built as hollow concrete cylinders with a 3-in. shell of 1:2 mortar, and filled with a 1:2:4 concrete after being sunk to place. As hollow cylinders, they avoided any coffer-dam work. The dock has a standard railroad track down each side over the outside columns, also an ordinary wooden fender system.

A still heavier concrete dock is now in course of construction at the same Navy Yard. It is 490 ft. long, 80 ft. wide, and is designed for a load of 600 lb. per sq. ft. The approach of the dock consists of a wooden pile structure, 210 ft. long, of triangular shape. As designed, the structure is supported by sixty-eight concrete columns, 4 ft. in diameter,

with an 11-ft. base (of the same type as in the dock just mentioned), 20 ft. from center to center athwart the dock, and 30 ft. longitudinally. The columns are capped by extremely heavy reinforced concrete beams which support a series of built-up structural beams, about 36 in. deep, carrying a thick concrete floor-slab. The side of the dock is cantilevered out 8 ft. beyond the columns. A standard railroad track runs over the center line of the outside rows of columns on each side of the dock. On account of the nature of the soil, some of the piers rest on sub-piling, others, nearer the shore, on hardpan.*

Vancouver.—The Great Northern Railroad has very recently completed an extensive reinforced concrete dock of the quay type, in connection with a new terminal it is building at Vancouver. The total width of the terminal dock is 302 ft., the concrete dock proper being 456 ft. long, and 50 ft. wide on each side of the terminal, the space between being a rock and earth fill, with a proper rip-rapped slope at its faces. Each concrete dock structure consists of nineteen reinforced concrete columns, 4½ ft. in diameter, 25 ft. from center to center, parallel to the face of the dock, resting on the rock stratum that underlies the location of the terminal. These columns support a heavy longitudinal concrete girder into which are tied heavy cross-girders, 12½ ft. apart. The cross-girders are cantilevered out beyond the longitudinal girder about 16½ ft., and their inboard ends rest on two driven concrete piles 34 ft. back from the center of the columns. Four concrete beams of suitable size are run longitudinally with the dock. The entire girder and beam structure supports a concrete slab. The railroad track is over the longitudinal girder running from column to column. The two parts of the dock are tied together by suitable concrete beams running across the interior fill. The terminal is considered to be one of the most substantial and up-to-date structures on the Pacific Coast. The dock is well protected by a wooden fender-pile steel-spring system.

Great Lakes.

Although extensive use of concrete has been made in some of the ports of Lakes Superior, Michigan, Huron, Erie, and Ontario, in the construction of massive ore-docks, it is not proposed to discuss that particular phase of the subject in this paper. These docks are of

* In the actual construction the built-up structural beams were replaced by reinforced concrete.

excellent design, some built on concrete piling, others on wooden piling or cribs. Concrete piles in fresh water are not subjected to the same deterioration as those in salt water. On the other hand, they have to withstand extremes of temperature, frost and ice, in addition to severe treatment, due to the heavy traveling machinery above them.

Some of these docks will be discussed, in order to bring out the important features of their special design.

Chicago.—One of the first attempts at using concrete for dock work on the Great Lakes was made at South Chicago, in the winter of 1898-99, in the rebuilding of an old wooden quay dock wall. The concrete structure is 1 680 ft. long, and consists of a heavy mass concrete stepped-back wall, 10½ ft. high, 8 ft. wide on top, and 18 ft. on the bottom, supported by a timber structure consisting of three rows of piling cut off 3½ ft. below mean water level, with longitudinal caps crossed by a heavy wooden grillage. A timber sheet-piling bulkhead under the face of the wall acts as a retainer for the slag fill. On one occasion the wall was rammed by a large steamer, but suffered absolutely no damage. The damage to the steamer, however, was rather extensive. A year or so later, a similar quay dock wall, 2 300 ft. long, was constructed at the same steel plant.

Due to the decay of a long wooden water-front bulkhead on the Chicago River, it became necessary to replace it. This was done by constructing a reinforced sheet-pile bulkhead almost ½ mile in length, capped by an I-shaped concrete beam, some 3 ft. wide and 5 ft. high. This beam rests on pre-moulded piles, which are 20 ft. apart and are secured to buttresses, also 20 ft. apart, which run back about 12 ft. from the sheet-piling, the land ends of the buttresses being supported by other piles. The buttresses are also tied back to deadmen some 35 ft. back from the wall. As the bulkhead has about 18 ft. of water along its front, vessels can dock alongside.

Marquette.—The most extensive reinforced concrete ore-loading docks on the Great Lakes are without doubt at Marquette, Mich., and were built in 1912. The substructure consists of a heavy concrete slab and facing-walls, similar to a channel beam placed on its back, 1 500 ft. long and 60 ft. wide, resting on wooden piling. Under the face of the walls there is a wooden sheet-piling bulkhead which retains the sand fill around the piles under the dock structure. The depth of

the concrete web is 3 ft., the walls that correspond to the flanges being 9 ft. high. The substructure supports a very massive reinforced concrete superstructure for loading ore into vessels, the whole dock being a very substantial and shock-resisting structure. A steel plate is worked along the face of the two concrete walls from 6 in. below the water level to 3 ft. above it, to prevent disintegration of the concrete due to frost and ice action at the water level.

Two Harbors.—At Two Harbors there is an ore-loading dock consisting of a steel superstructure supported by a mass concrete wall running along each face of the dock, and tied together with concrete beams at regular intervals. The concrete walls rest on wooden piles with a wooden sheet-piling bulkhead to retain the interior fill. The substructure is 1 400 ft. long and 52 ft. wide.

Detroit.—The concrete ore-loading dock at Detroit is 200 ft. long. It consists of three bearing walls of a T-rail shape, 9 ft. deep, running parallel to each other, the two outer walls being 28 ft. apart, the third or back wall being 173 ft. from the middle wall. The three walls rest on a double row of oak piling cut off 3 in. above the water level. Being capped by the concrete walls, no part of the wooden piling is exposed to the air. The outer wall stands in about 10 ft. of water, the middle wall in 3 ft., and the rear wall is far back on the dry land. The two outer walls are tied together by reinforced concrete beams, 5 ft. deep, at intervals of 10 ft., with cantilevered brackets on the water face of the outer wall opposite the tie-beams. The concrete deck-slab on top of the brackets is 12 in. thick, but only 6 in. thick between the two outer walls. The dock has a suitable wooden fender-pile system.

Between the middle and back walls there is a 12-in. reinforced concrete floor-slab, supported on a series of piles, 5 ft. from center to center each way, stated to have been designed for a load of 6 800 lb. per sq. ft., or 85 tons per pile. The ore floor is tied into the middle and back bearing walls opposite each cross-beam in the dock proper. The three walls support an ore bridge tower, used for unloading ore from vessels to cars on the track just behind the middle wall, or into the ore bin between the middle and back walls.

Toledo.—At Toledo, Ohio, there is an ore-unloading dock, consisting of a plain concrete wall running along the water-front, resting on wooden piles, and tied back to deadmen some 100 ft. in the rear. The

land is reclaimed back of the middle wall, a row of sheet-piling under the river wall retaining the earth in place. Parallel to the river there are three other walls to support the legs of the ore-unloading bridges which run on tracks placed on all four walls, spanning the four car tracks and the ore piles. The distance from the front wall to the inmost land wall is 419 ft.

Cleveland.—There is a full reinforced concrete ore-unloading dock at Cleveland, Ohio, completed in 1912 by the Pennsylvania Railroad. It consists of a reinforced concrete water-front wall, 985 ft. long, supported by a double row of pre-moulded concrete piles. The face wall is tied back by reinforced concrete tie-beams to another reinforced concrete wall supported by three rows of concrete piles about 81½ ft. in the rear. The tie-beams are 30 ft. apart, and rest on concrete piles 6 ft. apart. The space between the two walls, that is, under the concrete tie-beams, is back-filled with rip-rap, the whole mass being pumped full of sand. A concrete sheet-piling bulkhead under the front wall retains the fill in place. The dock has the usual wooden fender-pile protection. The tracks for four 17-ton Hewitt unloading machines run along the inner and outer walls. So far as the writer has been able to discover, this was the first concrete pile dock built on the Great Lakes.

Another ore-unloading dock at Cleveland consists of a reinforced wall of the same type as that at Detroit, the two structures having been designed by the same engineers. The cantilevered face of this wall is supported by brackets at intervals of 15 ft., with rear buttresses opposite them. The buttresses rest on a concrete base slab, the rear edge of which rests on a row of wooden sheet-piling, the base slab evidently being monolithic with the rest of the wall. The space between the sheet-piling and the wooden piles that support the concrete wall (11 ft.) is filled with slag. In general, the cross-section of the wall is dumb-bell shaped.

General.—In discussing the ore docks of the Great Lakes, it is necessary to consider the special and peculiar conditions under which they are operated, as they are designed to meet these conditions, and not *vice versa.* The larger portion of the freight carried on the Great Lakes is of heavy bulk form, *viz.,* ore and grain on the downward trip, coal on the upward trip. Thus ore and coal cargoes form the two heaviest items handled in bulk masses at the terminals. Nowhere in

the world is found such massive and modern machinery for the economical and expeditious handling of bulk freight as at the upper and lower ports of the Great Lakes. Consequently, such docks, wooden or otherwise, must not be judged by the type used along the seaboard of the Atlantic, the Gulf, or the Pacific.

It appears that the various types of reinforced concrete docks as worked out by the American engineers are far more numerous than in foreign practise, as is evident from the foregoing descriptions. Although the Atlantic Coast engineers seem to favor premoulded concrete piles, the Pacific Coast engineers apparently favor large concrete columns. Perhaps in time a typical American concrete dock will be designed or devised, as in the case of the long-standing type of wooden pile dock structures.

In the foregoing review of American reinforced concrete docks, an effort has been made to include each and every port wherein such types of docks exist, as well as to mention each and every dock already built, so far as the writer has been able to acquire sufficient information concerning them, in order that the exact situation as regards the development of reinforced concrete dock construction in America up to the present time may be known to all. If the writer has unintentionally omitted any such dock structure, he will be pleased to receive information relating to it.

The writer has endeavored to determine which was the first complete reinforced concrete dock constructed in the United States, but has been unable to do so. The concrete dock wall built at Chicago in 1898-99 appears to have been the first of its kind constructed on the Great Lakes. A study of the constructive dates of concrete pile or concrete column docks would indicate that such types began about 1905 or 1906. Still, it is not evident which was the first of such docks to come into existence, the whole development being a gradual evolution from a concrete-filled steel cylinder column, steel deck-beams, and concrete-slab type, as used in the Philippines by the United States Government in 1902.

Irrespective of the actual beginning of constructing reinforced concrete docks, it is generally conceded that Oscar F. Lackey, M. Am. Soc. C. E., Harbor Engineer of Baltimore, was among the first, if not the first, to blaze the way for the extensive use of reinforced concrete in dock construction in United States harbors.

Part III.—Failures of Reinforced Concrete Structures Subjected to Salt Water Action.

In discussing reinforced concrete docks, the fact that there have been failures among them must not be overlooked. In Massachusetts waters, north of Cape Cod, a number of serious cases of deterioration of concrete have been caused by the disintegrating effects of sea water, wave action, and frost, especially in Boston Harbor, where nearly all the concrete structures standing in sea water have been affected badly between high and low tide, the most notable instance of which is the concrete pier at the Charleston Navy Yard. Although that part of the pier which is constantly submerged has given but little trouble, the part exposed alternately to the sea and air has been seriously affected, many large pieces having broken completely away, making it self-evident that some other agent than the chemical action between cement and sea water was at work.

As is well known, winter temperatures on the whole eastern front of the New England Coast run far below zero. In Boston Harbor 12° below zero is not uncommon. In the same way that hard earth and porous rocks are broken up by frost action, permeable concrete in freezing water will gradually be destroyed between wind and sea, as the water which gets into the concrete simply exercises its natural expanding function in freezing, which *a priori* is detrimental to the concrete structure. It is generally admitted that the exterior concrete in these Boston structures, especially in the Navy Yard pier, has failed almost entirely from the effect of the alternate freezing and thawing with each tide during the winter, due to permeable concrete.

A number of failures similar to that already cited have occurred in Boston, the disintegration taking place in all cases between low and high tides. In the case of the Dover Street draw-bridge pier, built in 1894, the disintegration had extended 1.4 ft. into the pier at the end of 17 years, the greatest damage being just below high-tide level. The pier was built of 1:2:5 concrete, with a 1-in. plastered mortar facing. English Portland cement was used throughout. Whether the 1-in. facing mortar was expected to act as a water-proof shield to the interior concrete is not apparent. Evidently, it did not act thus, as might have been expected.

As all the concrete in these disastrous cases seems to have been placed in the wet, that is, the sea water was allowed to come in con-

tact with the concrete before it had become thoroughly cured and hardened, such results are not to be wondered at, for one of the axioms of a successful use of concrete in sea water is that it must be kept from contact with sea water for such a period of time as to enable it to become thoroughly hardened, especially that part between tides in freezing climates.

In several cases in Boston Harbor where the concrete was placed inside of a coffer-dam, or used in the form of pre-moulded, driven, concrete piles, the concrete does not seem to have been affected as in the other cases cited. These successful cases go a long way toward substantiating the truism that concrete, to be used successfully in sea water, especially in freezing water, must be made impermeable in the process of making, with full consideration given to the brand of cement used, the mixture, the sand, and stone (or gravel), the skilled labor of placing, as well as keeping it from contact with sea water until it has set and hardened sufficiently. It is very apparent, from a study of the method used in placing the concrete in the disintegrated structures in Boston Harbor, that that method was far from possessing the essential features necessary for a successful solution of the problem, viewed in the light of present-day knowledge.

In comparison with these Boston failures, it is fitting to state that at Dundee, Scotland, where the climatic conditions are said to be worse than at Boston, and where there is a rise and fall of the tide of about 12 ft., the combined action of the sea, waves, and frost has had no ill effect on the concrete docks in that harbor, the concrete piles of which were allowed to harden for 30 days before being put in place.

Another noted case of the destruction of concrete by frost and sea action is the large concrete sea wall along the water front of Lynn, Mass.—a massive concrete sea wall exposed to the pounding of the winter storms and seas. The steps to the beach in the front of this wall were destroyed to such an extent as to be hardly recognizable as steps. It might be of interest to state that this wall and some of the damaged structures in Boston harbor have apparently been repaired effectively by the cement gun process.

In reviewing these failures in Boston and vicinity, it is well to consider the results obtained in using concrete in another port subject to freezing and ice conditions, *viz.*, New York Harbor. In addition to freezing conditions, New York Harbor has to contend with a

strong tidal effect, which results in large solid ice floes and fields of broken ice moving back and forth with a tide of considerable velocity, ice floes of such size coming down the Hudson as at times practically to compel abandonment of all transfer traffic in that river. This is an effect from which Boston docks are perhaps free, as no large rivers flow into that harbor, the Charles being kept under control by the so-called Charles River Dam.

In discussing this additional handicap and destructive force at work on New York City's 8¼ miles of concrete sea walls, some of which have been in existence for 41 years, Charles W. Staniford, M. Am. Soc. C. E., Chief Engineer of the New York Department of Docks and Ferries, states:

"Up to the present time [August, 1911], no disintegration has been discovered that can be attributed to the existence of the structure in salt water. The concrete itself is in an admirable state of preservation, absolutely hard, and is undergoing no regular process of disintegration." * * * "this sea-wall which has been under construction * * * for 41 years, is at the present time an excellent piece of work and is subject to the same climatic conditions as all cities on the Northern Atlantic Coast with the attending ice, cold and rain characteristic of this latitude."

In many instances, parts of this wall above low water are faced with granite blocks. This is a noted example of what can be expected in the way of using mass concrete in sea water if properly made, though perhaps some repair work has been necessary in order to maintain the excellent condition of the wall.

In some of the earlier sections of this wall the concrete was placed "*en-mass*-in-site", but, since 1876, most of the wall has been built by the concrete block method. Only under specially favorable conditions is it possible to place concrete successfully *in situ* under (sea) water, as it becomes disintegrated "through the chemical action of the sulphate of magnesia on fresh concrete or through the resulting porosity of concrete due to the impossibility of tamping under water";* the viscosity and weight of the mass not being sufficient to produce such a dense material as obtained in block work.

To discuss an opposite case in New York Harbor, *viz.*, Dry Dock No. 2, New York Navy Yard, originally built of timber in 1890, the

* This subject is discussed further by the writer in an article entitled "Chemistry of Salt Water Cement," *Metallurgical and Chemical Engineering*, January and February, 1914.

history of which it is not necessary to relate here: In 1900 this dock was rebuilt, concrete being used very extensively. During 1913 a large sum was expended in repairing and replacing the concrete altars and floors. As it has been stated that the difficulties of using concrete in sea water have been so great at this yard as to indicate that this is not a permanent material for use in sea water structures, it would be of deep interest to learn the facts as to the chemical composition of the cement used, of the sand and stone, as to the mixture thereof, and the precautions taken in mixing and placing; also as to whether the dock is kept flooded when not in use, especially during the winter. If, as has been stated, the concrete "has deteriorated and disintegrated to such an extent that it was possible to use a pick and shovel in removing it", it is apparent that it was lacking in one or more of the essential features that are deemed absolutely necessary for a successful use of concrete in sea water structures.

Whether any of the concrete pile docks on the Great Lakes have shown any signs of deterioration due to frost action, the writer does not know, but trusts that some facts covering this question will be brought out in the discussion. As the water level is practically the same all through the winter, only a very short length of the pile would be affected, and not some 10 ft., as in Boston Harbor.

One of the first concrete docks built in San Francisco is said to have failed in part due to poor construction. The early method of building the concrete columns of San Francisco concrete docks was to use a wooden cylinder, strongly built, as a column form, into which, it has been stated, the concrete was poured, apparently without any attempt to pump out the cylinder. As long as the wooden cylindrical forms remained in place around the supposedly concrete column, the dock was pronounced a success. When the teredo had finally destroyed the forms, the columns began to collapse and the dock became a pronounced failure, because, in pouring the concrete, the heavier material —the stone or gravel—settled first, then the sand, and finally the cement. The result was that throughout the length of the concrete columns there were alternate layers of uncemented stone and sand, with the cement in between the sand of one batch and the stone of the following one. Concrete can be and is successfully dropped through a height of 50 ft.—and even up to 100 ft. in one noted case

in Arizona—but, if the receptacle into which it is dropped is full of water, disaster alone awaits the unfortunate engineer.

In another of the San Francisco docks, where wooden piles supported the concrete columns, the concrete was not carried down below the mud line a sufficient distance to prevent the teredo from destroying the piles below the concrete.

The question has been raised: Has any deterioration taken place in concrete structures standing in sea water in the harbors of the Southern States, where frost action is unknown. The most prominent concrete structure thus situated is the famous viaduct across the Florida Keys, built of Alsen cement, imported from Germany. It is possible that some of the members of this Society are in a position to give complete information regarding the action of salt water and the waves of the Gulf on this structure.

In order to guard against the disintegration (irrespective of its cause) of mass concrete placed *in situ* above low water, or to repair any damage that has been done, besides the cement gun process, various methods have been used, all based on the fundamental principle of using an impermeable material for the facing of the structure. Below low water, properly made block work has given most satisfactory results. Carefully made, fully cured pre-moulded concrete piles seem to resist the action of the sea and frost successfully. In Holland, hard, impermeable brick have been used to prevent any further damage to one of the breakwaters above low water. In England, the upper parts of massive breakwaters are mostly faced with granite or some other hard suitable stone. In Nova Scotia, both brick and pre-moulded blocks of concrete of small size were placed on the face of a concrete sea wall after the disintegrated concrete had been removed. A still more recent device is the use of hollow, vitrified, salt-glazed tile blocks filled with concrete after being put in place. Experiments thus far seem to have proved that:

"Vitrified salt-glazed tile is impervious to any deteriorating action of sea water, and has an effective structure against the battering of ice; it is so dense as to preclude the possibility of any water entering and freezing in it to the consequent destruction of the tile."

Though oiled concrete is being used as a water-proof material in certain cases, it is possible that the refuse, oil, gases, etc., discharged from certain classes of buildings, etc., might have a destructive effect

on the concrete foundation piles or other parts of the building, especially in sea water heavily charged with sewage. It is a well-known fact that concrete sewers will not perform their duty properly for any length of time unless they have a brick lining invert, over which flows the heavy sludge. In time of flood the surface water is so great as to dilute the sewage and prevent injurious effects. The writer would be pleased to hear opinions on this point, as it is possible that a destructive effect might have been caused by sewage in connection with one of the most seriously affected cases in Boston.

Although poor results seem to have attended quite a number of the reinforced concrete structures standing in sea water in America, the opposite appears to have been true in foreign countries. Still, a few failures are on record as having occurred in England and Germany, due mostly to permeable concrete.

Part IV.—Construction and Maintenance Cost of Reinforced Concrete Docks.

Cost of Construction.—To attempt any discussion of costs is always attended with danger to the one who does so, especially when such figures are "published cost". To lay the foundations for a discussion of this side of the question, the data in Table 2 have been collected from various publications, their real worth depending on the reliability of the published figures. The data cover some of the docks mentioned in Parts I and II.

The first cost of the various reinforced concrete dock structures at Port Talbot is stated to have been "no more than if they had been built of wood." Such a statement would not perhaps be true in the United States, on account of the large forests in this country.

Maintenance Cost.—A few figures covering the cost of maintenance of reinforced concrete docks in England are noted. In addition to what has already been said on the maintenance question, the cold storage wharf at Southampton is reported to have cost nothing to date for maintenance. On the other hand, the widened dock and the coal jetties are said to have shown considerable deterioration due to rusting of the steel, it having been improperly placed in these structures. It has been stated authoritatively that "while six to seven years is perhaps a rather short time in which to form any definite conclusions, the maintenance cost [of the above described Port Talbot docks] has been practically nothing."

The annual repair charges on the Purfleet coaling jetty (exclusive of the damage done at the time of the collision) for the first 9 years of its existence are stated to have been but $50 per annum, which, based on its cost, $60 000, is less than one-tenth of 1 per cent.

TABLE 2.—COST OF CONCRETE DOCKS.

Location.	Type.	Cost per square foot.
Pier No. 8, Puget Sound Navy Yard...	Concrete columns. Steel deck-beams. Concrete deck-slab.	$3.11
Naval Station, Philippines..............	Concrete columns. Steel deck-beams. Concrete deck-slab.	2.60
Balboa, Panama Canal..................	Concrete columns. Concrete beams. Concrete deck-slab.	3.28
Oakland, Cal...........................	Concrete piles. Concrete beams. Concrete deck-slab.	3.27
Brunswick, Ga..........................	Concrete piles. Wooden deck system.	1.40
Charleston Navy Yard, S. C............	Concrete piles. Wooden decking.	2.60
United Fruit Company, Panama.......	Concrete-protected wooden piles. Concrete deck-beams. Concrete slab.	2.13
Brooklyn, average of two docks........	Wooden piles. Wooden caps. Concrete deck-slab.	0.90

Table 3, from data published by the Chief Engineer of the Mersey Dock and Harbor Board in 1910, gives some very interesting results as respects the cost of annual repairs to six of the reinforced concrete docks at Southampton.

TABLE 3.—ANNUAL REPAIRS, SOUTHAMPTON DOCKS.

Dock.	Erected.	Cost.	Cost of repairs to date.	Average per year.	Annual percentage, based on original cost.	Remarks.
A	1899–1900	$24 000	$800	$80	0.0033	Deck for Cattle Wharf, Prince's Jetty (wood piles).
B	1904–06	63 500	375	75	0.0012	Floor or deck on Hennebique piles, Prince's Dock, West Quay.
C	1900	13 800	80	8	0.0005	Floor for wharf, Coburg Quay.
D	1901	3 700	25	3	0.0008	Floor for wharf, etc., Brunswick Half Tide Dock.
E	1908	17 500	......	....	....	Floor for wharf, etc., North Quay, Brocklebank Dock.
F	1908	163 500	......	....	....	Treble-story shed, South Quay, Sandon Dock.

Dock *A*. Subject to the effect of moist air arising from water below it.

Dock *B*. Subject to effect of moist air. Piles more or less submerged, according to water level.

Dock *C*. Complete reinforced shed and pile foundation; not sufficient time to form any conclusion.

If the results in Table 3 are true, it is no wonder that the English engineers report that their reinforced concrete docks cost nothing for annual repairs.

PART V.—GENERAL CONSIDERATIONS.

Opinions of Foreign Experts.—Although failures have accompanied the use of concrete in sea water and in the construction of reinforced concrete docks, it must be admitted that, if the Engineering Profession did not meet with a failure now and then, it would never acquire anything new, as it is through failures that it gains the most vital knowledge of engineering.

(*a*).—In discussing the question of concrete docks, a prominent New York engineer has stated that, of the large number of concrete docks which have come under his observation, the majority have been a success, though here and there he reports a failure due to poor construction and material, and not to defects in the design. It is authentically stated that the reinforced concrete docks at Southampton have shown no deterioration due to salt water action, except at the Southampton coal jetty. The engineers of the Liverpool Docks have been using concrete in connection with their work since 1872, apparently with great success. It has been stated on the best authority that in England the alternation of "dryness and wetness and fluctuations in temperature" does not appear to have affected reinforced concrete sea water structures adversely.

Mr. Henry Hunter, Chief Engineer of the Manchester Canal, England, states that "the concrete in the concrete lock built at Eastham is in better condition at the present date than the day it was deposited"; and adds that, covering an experience of more than 30 years of placing concrete in salt water, he has known no failures in such work where the concrete has been properly mixed and deposited.

In discussing the concrete docks of the Port Talbot Dock Company, Mr. William Cleaver has stated that:

> "While reinforced concrete requires extreme care, both in the choice of material and in the supervision of the workmanship, the results justify the extensive adoption of the material for dock work."

The exceptionally experienced dock engineer, Mr. Francis E. Wentworth-Shields, of the London and Southwestern Railway Company, has stated that "if great care is exercised in making and placing concrete,

an impermeable material will be obtained which can withstand the action of salt water." Mr. Shields has also said, "while many engineers were nervous about the life of reinforced concrete (for sea structures), he had observed that maintenance engineers were not so nervous as construction engineers." He is inclined to feel that there is nothing special to be feared respecting the life of reinforced concrete when used in marine work. Although, under certain circumstances, it is likely to deteriorate, he does not think it will do so from simply standing in sea water. He says that though in some cases deterioration has taken place at Southampton above low water, it has not done so below low water; that a 10-year-old reinforced concrete structure standing in sea water at Southampton is in perfect condition at the present date; and that, during the whole experience at Southampton, sea water does not seem to have produced any chemical or other deleterious action on the concrete.

Experiments by Mr. Baldwin-Weisman, in 1907, in England, on the permeability of concrete, show that, if it is well made, it is one of the most water-tight materials known, and that it rapidly becomes less and less porous when water is forced through it.

Mr. V. De Blocq von Keuffeler, in summing up the experience in using concrete for salt water structures in Holland, says:

> "A suitable mixture, very carefully manufactured, the use of a good brand of cement with trass, and setting in a moist atmosphere, are the most efficient means of ensuring the preservation of reinforced concrete in sea water."

Mr. I. Ho, one of Japan's expert harbor engineers, who used more than 1 200 mass-concrete blocks in one instance, none of which during a period of 10 years has shown the slightest signs of failure, states, "that whereas a good and proper cement is of consideration, the most important factor is the mode of fabrication."

In reviewing the successful experiences of some of England's leading authorities, the question of the chemical composition of the cement used does not appear to be given. Such information would be of great value, in order that engineers may know whether they use a cement especially manufactured for sea water concrete or simply the ordinary Portland cement, with or without puzzolana, trass, etc.

(*b*).—In discussing the deterioration of steel in reinforced concrete, by the action of sea water on ferro-concrete, provided the latter is properly made, Mr. C. S. Meiks, a prominent concrete engineer of England, says that such deterioration "is a negligible quantity." In

support of this contention he cites the experience at Southampton, stating "that the exposed steelwork on a pile end that had been in the sea for 8 years was much corroded", whereas the bars in the body of the concrete, on being cut open, were found to be quite free from any rust and as fresh as the day they were put into the pile.

In connection with the building of some of the earlier concrete dock structures at Southampton, it appears that parts or the whole of piles not used were allowed to remain on the beach or shore, exposed to sea water, for some 7 years. At the end of this period the exposed steel had been badly rusted and deteriorated, whereas the part which was embedded in concrete was found to be in fine condition, practically as good as the day it was placed in the concrete. Still, concrete piles lying on the beach are not in the same position as concrete piles subjected to shocks in a dock.

Though the first jetties, built 11 years ago in Southampton water, are in excellent condition at present, the steel in another jetty at the same location has deteriorated, due to electrolytic action.

It is generally accepted by all English authorities that no deterioration takes place in steel when well embedded in the concrete.

(*c*).—In speaking of reinforced concrete when used in marine work, Mr. Wentworth-Shields says "it will stand a wonderful amount of shock, and bending due to shocks, if a wooden fender is interposed." At a more recent date, Mr. Shields remarked: "On the other hand, reinforced concrete would not bear being knocked about by heavy ships, and where a structure was subjected to severe blows of that sort it was not easy to find anything better than timber, * * * but, when used at the right time and in the right place, reinforced concrete was a valuable material to dock engineers." From the leading position Mr. Shields occupies among the dock engineers of England, it would be of interest to know just what distinction there is between "a wonderful amount of shock" and "subjected to severe blows."

As an axiom: whatever system or design is adopted for a reinforced concrete dock, in no manner whatsoever should a vessel be allowed to rub against the main piling of the dock. The dock should always be protected by a system of fender-piles.

(*d*).—As respects the resistance of a concrete pier or dock, when under such treatment as the Purfleet pier was at the time it was rammed, in 1904, Mr. Meiks, engineer in charge of its construction,

states that "the vibration was so great at the time of the collision that they thought the entire pier would collapse, but that its elasticity was most satisfactory, due no doubt to its horizontal concrete decking." Mr. Meiks also says that the vibration of a concrete pier supported on piles is nearly as great as in a pier made of timber piling; but this has no particular effect on the structure, judging by the experience gained with the Purfleet pier at the time it was rammed.

In speaking of the Port Talbot docks, Mr. A. E. Carey has stated:

"If the structure [reinforced concrete dock] was properly designed and built, its stability and life were assured, the only serious drawback being the difficulty of repairing damage due to collision."

But how often do collisions happen?

The Chief Engineer of the Port of London, Mr. Bryson Cunningham, recently stated that the art of building reinforced concrete docks in England has attained a degree of perfection greatly in advance of early experimental work. If their early experimental docks are still doing good service, will not their more recent docks become structures of an engineering and commercial success, thus justifying the American engineer in recommending concrete docks as long as they are built in a manner to guarantee impermeability and non-deterioration of the concrete?

Concrete Breakwaters.—As the application of reinforced concrete to dock construction has been developed almost entirely since 1900, some of the most conservative engineers may not feel that sufficient time has elapsed to judge correctly as to the merits of using cement and placing concrete structures in sea water, and as to the advisability of adopting reinforced concrete as a coming type of dock structure. As respects the first point, the prolonged and successful use of mass concrete by foreign countries in breakwater, tidal, and graving dock work would in itself appear to be sufficient answer to all such skepticism.

Though an extensive treatise might be written on concrete breakwaters and their construction, including shore protection, those phases of the use of concrete in sea water are supplementary to the principal subject of this paper. There is apparently hardly a leading seaport or a maritime nation outside of the United States that has not made a wide, extensive and successful use of mass and reinforced concrete in the development of harbor and shipping facilities.

Surprising as it may seem, a number of large concrete breakwaters

have been in existence in Japan for more than 18 years. As a matter of fact, the use of concrete in the harbor work of Japan is far in advance of American practice, being apparently on the same high level as in England and other European countries.

Mass concrete has been used very extensively in Belgium and Holland for sea water structures for years, and has given the best of results. In fact, some of the concrete sea walls in the latter country, built in 1867-77 and earlier, are so old as to be called ancient, and have as yet shown no signs of being affected by the action of sea water. Perhaps such a statement needs to be qualified, because some of the principal harbors of Holland are some distance from the sea, and are in fresh or brackish water.

Concrete was used by the Romans and Carthagenians in ancient times. Though it fell into disuse for many centuries, it came into use again in 1840-50. To this day, sea walls built of puzzolana and lime cement by the Romans are in existence in Italy.

The Italian engineers report that Portland cement concrete with an addition of one-eighth to one-tenth by volume of puzzolana gave no signs of disintegrating in salt water, even after an exposure of 30 years in the harbors of Genoa, Civita Vecchia, Naples, etc.

At several places on the Italian coast concrete-faced breakwaters have been in existence since 1880, in most exposed positions, costing but little for repairs and maintenance, though subject to the high seas and heavy blows of the Mediterranean.

English engineers were using mass concrete in their tide locks and in the construction of massive breakwaters along the coast of England as far back as 1871, if not earlier. The fact that they have continued to use it more extensively each year, even to building vast reinforced concrete structures standing in sea water during the past 15 years, would appear, in spite of some failures, to be sufficient answer to any doubts the American engineer may entertain on the subject.

At Colombo, India, a concrete breakwater, finished in 1885, showed no failures above or below water at the end of 22 years. As stated above, some of the massive concrete breakwaters of the world have a hard stone facing, or are built of concrete blocks, with or without a stone facing.

The reasons that enable the foreign engineers to accomplish such lasting results with concrete is no doubt due to the fact that they, to-

gether with the foreign chemists and cement manufacturers, long ago learned the secret and acquired the art of manufacturing and using concrete in sea water structures. Though the American engineer excels the foreign engineer in certain lines of his profession, it must be admitted that, so far as using cement, and hence concrete, in sea water structures, the engineers of the leading European countries and of certain parts of South America are many years in advance. The American cement manufacturer, the chemist, and the harbor development engineer cannot long remain in such a position without reflecting on their ability as experts in their respective lines of work in the minds of their foreign contemporaries.

Part VI.—Conclusion.

In view of the marked success obtained by foreign engineers in the use of concrete for sea water structures, when the execution of such undertakings has been placed in the hands of intelligent, skilful, and experienced men, the American engineer who denies the possibility of making a successful use of concrete for structures standing in sea water puts himself in a questionable position. He thereby confesses either his lack of a world-wide knowledge on the subject, or his inability to carry out properly such classes of construction work to the same successful conclusions as his foreign contemporaries have been doing for years past. Such a confession would seem to indicate a lack of foresight and ultra-conservatism as respects the use of cement subject to sea water conditions, on the part of the American cement manufacturer, chemist, and concrete engineer.

The American engineer who assumes a skeptical attitude toward the practicability and commercial success of reinforced concrete docks has standing before him as silent testimony of their worth and practicability such a vast number of foreign reinforced concrete docks—several about 20 years old and yet in excellent condition, with still more massive structures being built each year, some of them costing less than wooden docks, if reports are true, and most of them saving their owners large sums annually on account of their low cost of maintenance and repairs, with no rebuilding, as with our 15-year creosoted wooden pile docks—that the grounds on which he stands become somewhat untenable. It is true that all American cements are not as yet wholly suitable for sea water purposes, and perhaps equally true that each and every American concrete engineer has not hitherto insisted

on the proper placing of concrete in salt water structures, not fully realizing the importance of the fundamental principles of the use of concrete in sea water, due to the hitherto limited call for such types of structures in America, lumber having been so plentiful and cheap.

Though reinforced concrete docks have been in existence for more than 15 years, and operated successfully, the same old theorems are still put forth in opposition to them and to the practical experience gained during this period. Possibly these same theories will continue to be advanced against the use of concrete in dock work and other sea structures by the most conservative of our leading engineers, though others, guided and profiting by the experience already gained in such uses of concrete, will continue to expend large sums of money in the further development of such structures.

From a prolonged study of reinforced concrete dock construction, as carried out in foreign countries, it would appear that, in spite of early doubts and skepticism, the success in the use of reinforced concrete in dock work obtained by foreign engineers has swept away all such doubts and skepticism. If these are not facts, why are foreign countries and certain ports in America, including the United States Government, expending vast sums of money in building reinforced concrete docks and in other uses of concrete in harbor development and sea protection work; all "in spite of prejudices which leading engineers [psychologically] have against any new type of construction."

Although the average American contractor may look upon concrete as just so much cement, sand, and stone, or gravel, to be thrown together and dumped into the forms in the quickest possible time, without any regard for the fundamental principles underlying reinforced concrete construction, a commercial proposition purely, such an application of reinforced concrete to dock work will most certainly spell disaster long before the structure is completed.

In spite of its apparent simplicity on dry land, the use of reinforced concrete in dock work calls for more than mere brawn and muscle. It is a class of construction work especially adapted to the broad knowledge, experience, and deep study of the trained engineer in association with an organization well skilled in the handling of concrete in sea water structures—"a field of engineering in which reinforced concrete will prove to be the most permanent and economical, as it has in the building of bridges, etc."

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

FLOOD FLOWS.

Discussion.*

By Messrs. G. B. Pillsbury and Weston E. Fuller.†

G. B. Pillsbury, Assoc. M. Am. Soc. C. E. (by letter).—Accepting the premise that the magnitude of a flood is due to the combination of an unlimited number of accidental causes, the probability of the occurrence of a flood exceeding a given magnitude should follow the well-established laws of probability as deduced in the theory of errors of observations. Under these laws the probability of an annual flood less than a given value, Q', is given by the formula: Mr. Pillsbury.

$$P = \frac{h}{\sqrt{\pi}} \int_0^k e^{-h^2 x^2} \, dx \quad \text{(1)}$$

and of an annual flood greater than Q'

$$P = \frac{h}{\sqrt{\pi}} \int_k^\infty e^{-h^2 x^2} \, dx = \frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2} \, dt \quad \text{(2)}$$

where $k = Q' - Q$ (Ave.) and $t = h\,k$.

The value of h is given by the formula :

$$h = \sqrt{\frac{n-1}{2 \Sigma v^2}}$$

where n is the number of floods observed and Σv^2 is the sum of the squares of the residuals of the observed values with relation to the average yearly flood.

The reciprocal of P, Equation 2, will then represent the period of years in which, in the long run, an annual flood equalling or exceeding Q' may be expected.

* Continued from January, 1914, *Proceedings*.

† Author's closure.

Mr. Pillsbury.

The usual probability tables give the value of the function, $\frac{2}{\sqrt{\pi}} \int_0^t e^{-t^2}\, d\, t$, from which the function, $\frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2}\, d\, t$, in Equation 2 can be computed from the relation,

$$\frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2}\, d\, t = \frac{1}{2} \left(1 - \frac{2}{\sqrt{\pi}} \int_0^t e^{-t^2}\, d\, t\right).$$

The author, however, has taken the average value of the annual floods equalling or exceeding the given value as his measure of the expected flood, and not the flood of magnitude which would be derived from Equation 2.

The mathematical expression for this average or mean of floods exceeding Q' is

$$Q = Q\,(\text{Ave.}) + \frac{\frac{h}{\sqrt{\pi}} \int_k^\infty x\, e^{-h^2\, x^2}\, d\, x}{\frac{h}{\sqrt{\pi}} \int_k^\infty e^{-h^2\, x^2}\, d\, x}$$

$$= Q\,(\text{Ave.}) + \frac{1}{h}\; \frac{\frac{1}{2\sqrt{\pi}}\, e^{-t^2}}{\frac{1}{\sqrt{\pi}} \int_t^\infty e^{-t^2}\, d\, t} \quad \ldots\ldots (3)$$

$$= Q\,(\text{Ave.}) + \frac{1}{h}\, M \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

It is obvious that this mean value is somewhat greater than its inferior limit.

Table 39 shows the relation between the time, T, considered (or the reciprocal of P in Equation 2), and the value of M (Equation 4).

TABLE 39.

t	$T = \frac{1}{P}$	M
0.0	2.	0.564
0.2	2.57	0.697
0.4	3.50	0.841
0.6	5.05	0.994
0.8	7.77	1.155
1.0	12.72	1.32
1.2	22.32	1.49
1.4	42.02	1.67
1.6	84.88	1.85
1.8	185.2	2.05
2.0	434.8	2.25
2.2	1 000.	2.34

Taking the floods of Tohickon Creek, as given by the author in Table 4, the computation of the mathematical values of average maximum flood, under the theory of probabilities, is shown in Table 40. Mr. Pillsbury.

TABLE 40.

Date.	Annual flood.	Ratio to average.	V	V^2
1884	4 379	1.06	+ 0.06	0.0036
5	3 664	0.89	— 0.11	0.0121
6	5 359	1.30	+ 0.30	0.0900
7	2 544	0.62	— 0.38	0.1444
8	3 493	0.85	— 0.15	0.0225
9	4 714	1.15	+ 0.15	0.0225
1890	2 942	0.71	— 0.29	0.0841
1	2 858	0.69	— 0.31	0.0961
2	3 158	0.76	— 0.24	0.0576
3	2 994	0.73	— 0.27	0.0729
4	8 650	2.10	+ 1.10	1.2100
5	3 857	0.93	— 0.07	0.0049
6	6 515	1.59	+ 0.59	0.3461
7	3 683	0.89	— 0.11	0.0121
8	4 160	1.01	+ 0.01	0.0001
9	3 222	0.78	— 0.22	0.0484
1901	4 089	0.99	— 0.01	0.0001
2	5 958	1.45	+ 0.45	0.2025
3	4 968	1.21	+ 0.21	0.0441
4	4 395	1.06	+ 0.06	0.0036
5	4 175	1.01	+ 0.01	0.0001
6	3 200	0.78	— 0.22	0.0484
7	4 120	1.00	0	0
8	2 770	0.67	— 0.33	0.1089
9	3 050	0.74	— 0.26	0.0676

Average = 4 117 $\Sigma v^2 = 2.7027$ $h = 2.11$.

The probable average maximum floods, according to the theory of probabilities, and the observed averages as determined by the author, are plotted on Fig. 14.

Table 41 and Fig. 15 show the floods of the Connecticut River at Hartford for 70 years, beginning 1843. The observed average maximum floods shown on the figure are computed in Table 42.

The floods of the Allegheny River at Kittanning for a 41-year period are tabulated in Mr. Knowles' discussion, Table 33. The value of h, as derived from these data is 2.03. The curve of probable average maxima, the author's curve, and the observed values, are plotted in Fig. 16.

A very cursory examination of the figures shows that, for individual rivers, the observed average maxima differ widely, both from the author's formula and from their theoretical values under the theory of probability. This, indeed, is not extraordinary. The highest of the plotted observed figures is but a single flood, and not an average. The first few of the succeeding observed averages are based on very limited data, and may be expected to differ from their theoretical values in the long run. In the case of Tohickon Creek, the extraordinarily

Mr. Pillsbury.

TABLE 41.—YEARLY FRESHETS IN THE CONNECTICUT RIVER AT HARTFORD.

Year.	Maximum gauge.	Max. yearly flood.	Ratio.	V	V^2
1843	27.2	175 000	1.55	+ 0.55	0.3025
4	19.5	97 000	0.86	— 0.14	0.0196
5	19.0	93 000	0.83	— 0.17	0.0289
6	18.7	90 000	0.80	— 0.20	0.0400
7	21.0	110 000	0.98	— 0.02	0.0004
8	15.5	68 000	0.60	— 0.40	0.1600
9	17.5	81 000	0.72	— 0.28	0.0784
1850	20.7	107 000	0.95	— 0.05	0.0025
1	14.5	61 000	0 54	— 0.46	0.2116
2	23.1	130 000	1.15	+ 0.15	0.0225
3	20.5	106 000	0.94	— 0.06	0.0036
4	29.8	205 000	1.82	+ 0.82	0.6724
5	15.0	64 000	0.57	— 0.43	0.1849
6	23.3	132 000	1.17	+ 0.17	0.0289
7	19.5	97 000	0.86	— 0.14	0.0196
8	12.2	48 000	0.43	— 0.57	0.3249
9	26.4	166 000	1.47	+ 0.47	0.2209
1860	16.0	71 000	0.63	— 0.37	0.1369
1	21.5	115 000	1.02	+ 0.02	0.0004
2	28.7	192 000	1.70	+ 0.70	0.4900
3	15.0	64 000	0.57	— 0.43	0.1849
4	17.2	79 000	0.70	— 0.30	0.0900
5	24.7	147 000	1.30	+ 0.30	0.0900
6	20.5	106 000	0.94	— 0.06	0.0036
7	20.0	101 000	0.90	— 0.10	0.0100
8	21.5	115 000	1.02	+ 0.02	0.0004
9	26.5	167 000	1.48	+ 0.48	0.2304
1870	25.3	154 000	1.37	+ 0.37	0.1369
1	18.5	89 000	0.79	— 0.21	0.0441
2	19.7	99 000	0.88	— 0.12	0.0144
3	20.9	109 000	0.97	— 0.03	0.0009
4	23.8	137 000	1.22	+ 0.22	0.0484
5	18.4	90 000	0.80	— 0.20	0.0400
6	21.9	121 000	1.07	+ 0.07	0.0049
7	22.8	130 000	1.15	+ 0.15	0.0225
8	23.9	142 000	1.26	+ 0.26	0.0676
9	21.4	116 000	1.03	+ 0.03	0.0009
1880	15.0	66 000	0.59	— 0.41	0.1681
1	16.4	75 000	0.67	— 0.33	0.1089
2	14.7	64 000	0.57	— 0.43	0.1849
3	20.5	108 000	0.96	— 0.04	0.0016
4	21.9	121 000	1.07	+ 0.07	0.0049
5	18.1	88 000	0.78	— 0.22	0.0484
6	21.7	119 000	1.06	+ 0.06	0.0036
7	22.5	127 000	1.13	+ 0.13	0.0169
8	19.4	98 000	0.87	— 0.13	0.0169
9	15.6	70 000	0.62	— 0.38	0.1444
1890	16.0	73 000	0.65	— 0.35	0.1225
1	19.8	102 000	0.90	— 0.10	0.0100
2	....	....	...	....	......
3	24.0	143 000	1.27	+ 0.27	0.0729
4	13.8	59 000	0.52	— 0.48	0.2304
5	25.7	161 000	1.43	+ 0.43	0.1849
6	26.5	170 000	1.51	+ 0.51	0.2601
7	20.8	111 000	0.99	— 0.01	0.0001
8	21.2	114 000	1.01	+ 0.01	0.0001
9	22.0	122 000	1.08	+ 0.08	0.0064
1900	23.4	136 000	1.21	+ 0.21	0.0441
1	26.4	169 000	1.50	+ 0.50	0.2500
2	25.5	159 000	1.41	+ 0.41	0.1681
3	23.3	135 000	1.20	+ 0.20	0.0400
4	19.2	97 000	0.86	— 0.14	0.0196
5	24.0	143 000	1.27	+ 0.27	0.0729
6	19.8	102 000	0.90	— 0.10	0.0100
7	20.7	110 000	0.98	— 0.02	0.0004
8	18.1	88 000	0.78	— 0.22	0.0484
9	24.7	150 000	1.33	+ 0.33	0.1089
1910	19.4	99 000	0.88	— 0.12	0.0144
1	14.8	65 000	0.58	— 0.42	0.1764
2	20.5	108 000	0.96	— 0.04	0.0016
3	26.0	165 000	1.46	+ 0.46	0.2116

Average = 112 700 $\Sigma v^2 = 6.6912$ $h = 2.27$.

Mr. Pillsbury.

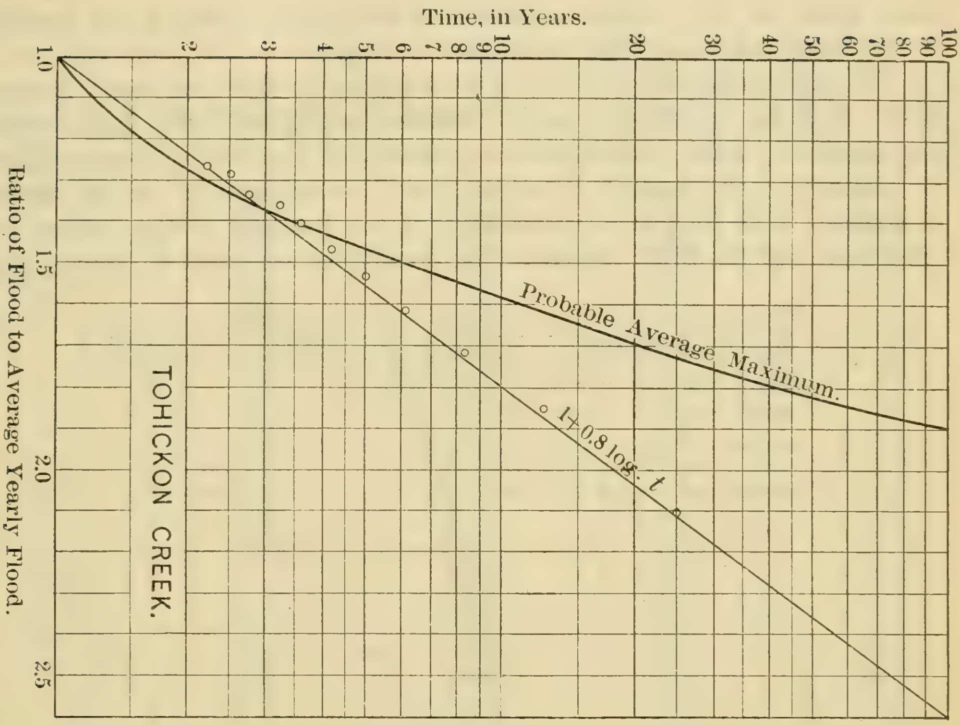


Fig. 14.

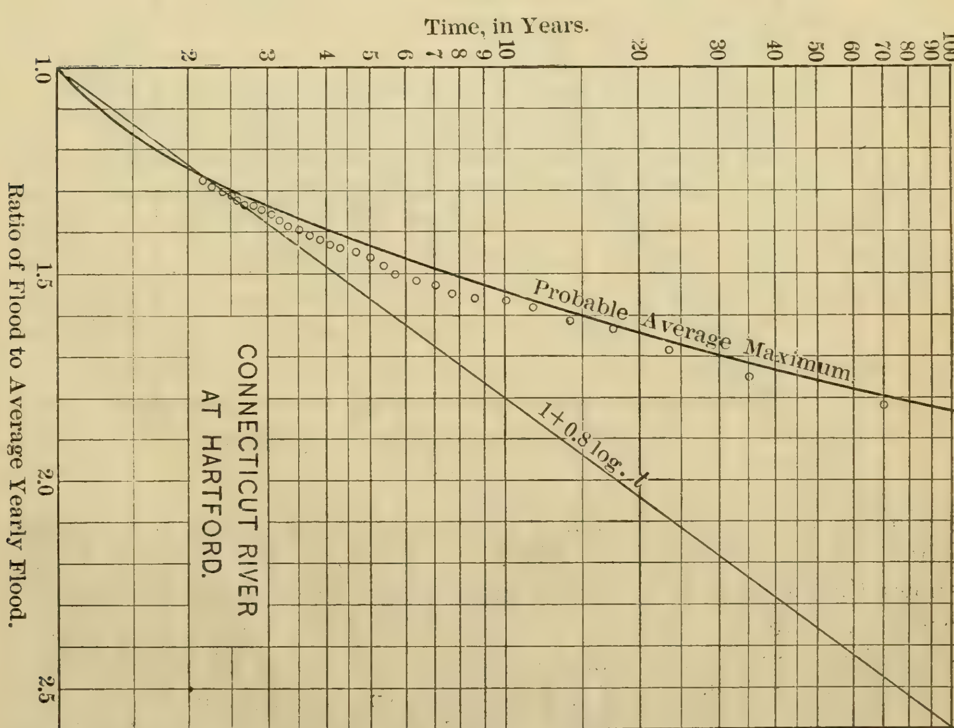


Fig. 15.

Mr. Pillsbury. large value of the maximum observed flood has a very great bearing in the large values of the next lower averages.

The application of the law of probability to the maximum average floods, as given by the writer, is intended merely as a side light on the discussion, it being excessively cumbersome for useful computation. It appears to the writer, however, that the application of the theory to streams with long records shows that the formula of the author will often give far too high values to the flood to be expected in such a long

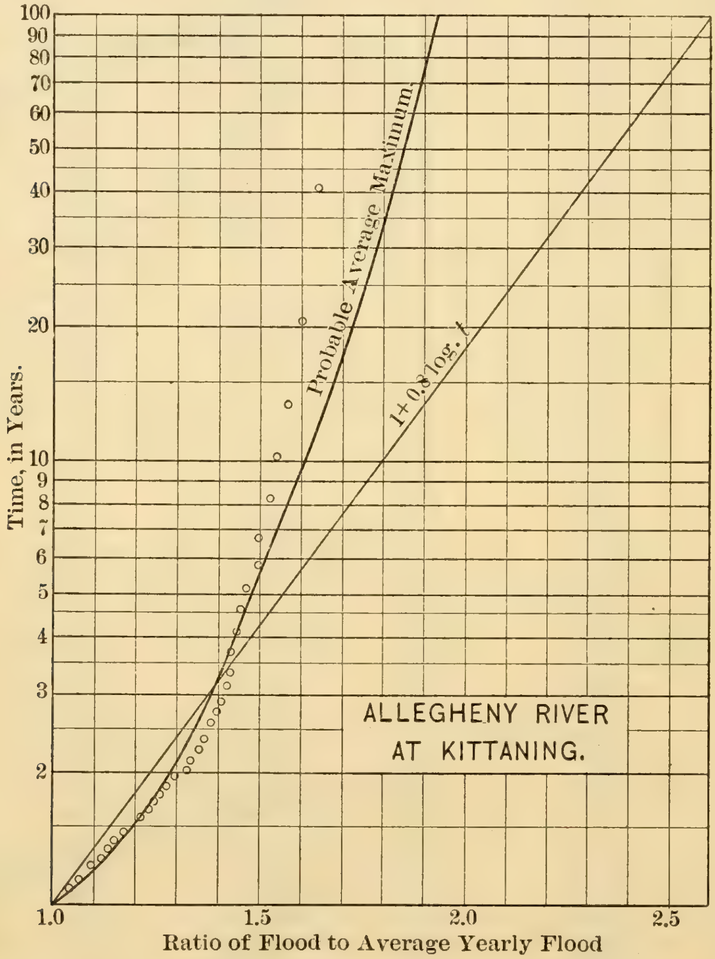


FIG. 16.

period as 1 000 years—which is another term for saying ever expected. Thus, in the Connecticut River at Hartford, the author's formula shows that a flood of 3.4 times the average, or of about 380 000 cu. ft. per sec., is to be expected in a period of 1 000 years. Dropping the somewhat cumbersome average maxima, the theory of probabilities indicates that the chance of a flood twice the average, or 225 000 cu. ft. per sec., is but one in 1 000. Would an engineer be justified in constructing works to care for more than, say, 250 000 cu. ft. per sec.?

TABLE 42.

Mr. Pillsbury.

No. of flood.	Ratio to average.	Summation.	Summation ÷ No. of flood.	Time, in years.
1	1.82	1.82	1.82	70.0
2	1.70	3.52	1.76	35.0
3	1.55	5.07	1.69	23.3
4	1.51	6.58	1.64	17.5
5	1.50	8.08	1.61	14.0
6	1.48	9.56	1.59	11.6
7	1.47	11.03	1.57	10.0
8	1.46	12.49	1.56	8.75
9	1.43	13.92	1.55	7.8
10	1.41	15.33	1.53	7.0
11	1.37	16.70	1.52	6.4
12	1.33	18.03	1.51	5.8
13	1.30	19.33	1.49	5.4
14	1.27	20.60	1.47	5.0
15	1.27	21.87	1.46	4.7
16	1.26	23.13	1.44	4.4
17	1.22	24.35	1.43	4.1
18	1.21	25.56	1.42	3.9
19	1.20	26.76	1.41	3.7
20	1.17	27.93	1.40	3.5
21	1.15	29.08	1.39	3.3
22	1.15	30.23	1.38	3.2
23	1.13	31.36	1.36	3.0
24	1.08	32.44	1.35	2.9
25	1.07	33.51	1.34	2.8
26	1.07	34.58	1.33	2.7
27	1.06	35.64	1.32	2.6
28	1.03	36.67	1.31	2.5
29	1.02	37.69	1.30	2.4
30	1.02	38.71	1.29	2.3
31	1.01	39.72	1.28	2.26

Mr. Fuller.

WESTON E. FULLER, M. AM. SOC. C. E. (by letter).—During 1913, and since this paper was written, an unusually large number of disastrous floods have occurred. It is natural that the effect of these great floods on the frequency relation should be discussed and that questions should be raised as to the applicability of the proposed relations to the rivers on which these floods occurred. Several who have discussed the paper have suggested the establishment of different frequency relations for individual streams or for groups of streams.

The writer, in presenting the paper, stated that the formulas proposed were intended to serve "as a frame work on which to arrange the data in an orderly manner, so that they can be better understood and more readily used." With this object in view, formulas were derived which expressed the average frequency relation for the floods which have occurred on many rivers widely distributed over the country. Tables were presented giving values of the coefficient, C, as obtained from the average yearly flood for such rivers as had been observed a sufficient number of years to give an approximate idea of the size of this average. In these tables were also included other values of C, obtained by reducing the larger floods by the use of the proposed formula. These latter values of C indicate how the actual

Mr. Fuller. floods which have occurred on the rivers have agreed with the proposed formula. The close agreement, for most of the rivers, of these two sets of coefficients indicates that the relation is a general one. During the course of the study on which the paper was based, plottings were made for many individual streams and for groups of streams in different sections of the country. The effect of the few larger floods on the plottings for individual streams was so great that the writer concluded that the relation indicated by them was less accurate than the average relation. Plottings for different sections of the country varied to some extent. If streams in partly arid sections are excluded, the variation of the coefficient of log. T in the formula is from 0.7 to 1. In the extreme cases the number of streams and the length of the records were too short to furnish proof sufficiently strong to justify any change from the general relation. That different frequency relations do exist for individual streams and for different sections of the country is probable, and, as more data become available, such relations may be established.

Great floods which have occurred on the streams in Ohio since the paper was written indicate either that in this section of the country great floods occur more frequently than in other sections, or that these floods were very extraordinary ones. It is unfortunate that so few data are available for floods on the rivers of Ohio, Indiana, Illinois, and, in fact, the streams in all the States along the Mississippi Valley. Floods on streams in adjoining sections indicate that the relation is similar to the average one proposed in the paper. The floods of 1913 in Ohio were caused by rainfall of extraordinary intensity, considering the large area covered, occurring under conditions favorable for great floods. That these conditions were extraordinary is certain, but whether such conditions occur more frequently in this section than elsewhere only the future will show.

Floods of 1913.—Although data are not yet available for all the great floods which occurred during 1913, records of those on some rivers in Ohio and New York have been published. Among those of particular interest are the floods on the Miami River at Dayton, Ohio, the Scioto and Olentangy Rivers at Columbus, Ohio, and the Hudson River at Mechanicsville, N. Y. The data for these rivers, revised to include the records of 1913, are given in Table 43.

The value given for the ratio between the flood of 1913 and the average flood for the streams at Columbus corresponds to a value of T equal to about 1 400. As there are more than 1 400 records of floods on different rivers for which the ratios to the average flood are now available, the occasional occurrence of such a flood is not surprising.

For the Miami River at Dayton, Mr. Morgan expresses the opinion that the coefficient, C, in the writer's formula should not exceed 50. If this were the proper value for C, the 1913 flood on the Miami

TABLE 43.

Mr. Fuller.

Name of stream.	Location of station.	Catchment area, in square miles.	Flood Flows, in Cubic Feet per Second.		Period of observation, in years.	Ratio of maximum flood to average flood.	Probable value of *C*.
			Average yearly.	Floods of 1913.			
Upper Scioto...	Water-works dam...	1 032	19 300	68 000	16	3.52	75
Olentangy......	Columbus, Ohio.....	520	14 500	51 000	16	3.52	97
Lower Scioto...	Columbus, Ohio.....	1 570	33 800	119 000	16	3.52	94
Miami..........	Dayton, Ohio........	2 450	50 000*	246 000*	21	4.92	90
Hudson.........	Mechanicsville, N. Y.	4 500	44 500	108 000	23	2.42	53

* Represents rate at maximum stage.

The data for the floods at Columbus are from the "Report on Flood Protection for the City of Columbus" by John W. Alvord and C. B. Burdick, Members, Am. Soc. C. E.

The data for the maximum flood at Dayton are from the discussion by Arthur E. Morgan, M. Am. Soc. C. E.

The data for the maximum flood on the Hudson are from *Engineering Record*, April 12th, 1913, "Effect of Recent Floods on New York Streams" by R. E. Horton, M. Am. Soc. C. E.

River would be relatively very greatly in excess of any flood which we have known on other rivers. The data on which to base the value of C for the Miami are not very satisfactory. In the papers of the U. S. Geological Survey there are 4 years, 1906 to 1909, inclusive, for which floods at Dayton are recorded. An average of these four floods would indicate a value of C of about 86. The best indication of the average flood at Dayton is that deduced from the gauge heights of the U. S. Weather Bureau, which are available for 21 years. As the rating curve for large floods is indefinite, it seems best to obtain the average flood by means of the median gauge height during this period. This has been 11.9, corresponding to a flood of about 39 000 cu. ft. per sec., according to the rating by the U. S. Geological Survey in 1906. During the last 10 years the median gauge height has been 13.25, which corresponds to a flood of about 45 000 cu. ft. per sec. From other rivers it has been found that the median flood is usually less than the average by about 10 per cent. On this basis the average yearly flood on the Miami at Dayton, on the 24-hour basis, under present conditions, is probably at least 45 000, and the value of C is at least 90. The corresponding maximum rate of flow would be 50 000 cu. ft. per sec., or more.

During the past 20 years many changes have been made in the channel of the Miami, such as encroachment on the river channel in cities, and the construction of bridges with long approaches of solid embankment, which greatly reduce the waterway. It seems probable that the coefficient of C for the Miami River at the present time is different from what it was a number of years ago. Mr. Morgan thinks that the Miami should have a comparatively low coefficient of flow on account of the comparatively small average slope of the water-shed. Steep slopes on the upper branches with smaller slopes on the main

Mr. Fuller. river and lower branches are conditions which may produce larger floods than uniform steep slopes. A comparison of the coefficients for different rivers of the same general character fails to indicate any great effect from the average slope of a river. The writer suggests that the relation between the slopes on different parts of the river and on its branches is of greater importance in determining the flood-producing capacity than the average slope.

The maximum flood on the Miami in 1913 may have been very different in magnitude from the flood which would have occurred if the channel had been unobstructed. The large number of embankments thrown across the valley, with inadequate waterways, provided temporary storage which tended to retard the earlier run-off until the branch streams poured in their maximum flood flows. The subsequent failure of these embankments releasing the stored waters, probably increased the flood materially.

Taking these matters into consideration, together with the great uncertainty of gauging such a flood, it seems probable that the recorded flood flow may have been considerably in excess of the flood which would have come under natural conditions. On the whole, the writer believes that the Miami flood of 1913 was not greater than five times the average yearly flood under present conditions, and that it may have been much less. At all events, the flood was a very exceptional one, and must be given careful consideration in determining the probable floods on our rivers, particularly those in the central part of the country.

The flood on the Hudson River in 1913 is an interesting one, from the standpoint of what may happen on other rivers. The maximum flood on the Hudson, for which records for many years are available, was, previous to this year, much less than what would normally be expected on the basis of its average flood. The flood of 1913, however, not only reached the normal maximum flood for the period of record, but exceeded it by a considerable quantity. There are many other rivers on which the maximum recorded flood is much less than the flood indicated by the average relation. That much greater floods will occur on some of these rivers within a comparatively short period seems assured. As an instance, the greatest flood on record on the Ohio at Wheeling and at Pittsburgh is equivalent to a flood which may normally be expected in a period of not more than 15 years. If a flood of the relative size of that on the Hudson occurs on the Ohio, it will mean a flood of more than 600 000 cu. ft. per sec. at Pittsburgh, or a flood from 4 to 5 ft. higher than the highest now on record.

Reliability of Frequency Relation as Established from Records on One Stream.—Messrs. Knowles, Horton, and Pillsbury give frequency relations for individual streams, which differ from the average relation

proposed by the writer. To show how unreliable such relations may be, Figs. 17 and 18 are presented, which show how greatly one or two of the larger floods affect the relation, even when derived from the longest records available for any stream. Mr. Fuller.

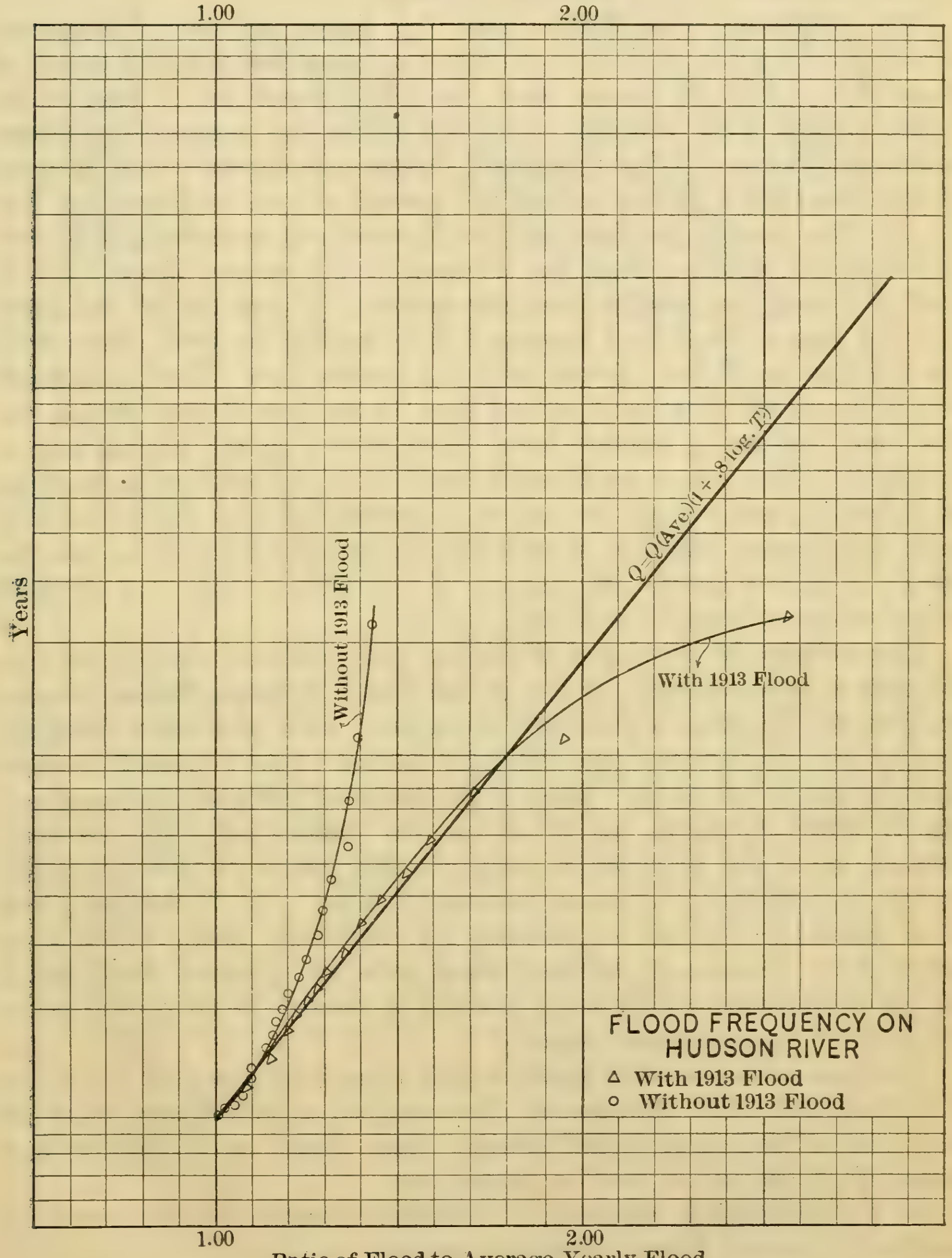


FIG. 17.

On Fig. 17 two curves illustrate the effect of a single great flood. The flood of 1913 on the Hudson is the greatest of which we have record. The effect of this flood is sufficient to change the data so that

Mr. Fuller. the indications are just the reverse of what they were before the flood occurred. In other words, prior to 1913, the Hudson River records indicated that great floods occurred less frequently than on the average river. By including this flood in the data, the indications are that such floods occur with a greater frequency.

Mr. Pillsbury contributes data for floods on the Connecticut River for 70 years, and Fig. 15 shows a frequency relation based on these data. Fig. 18 shows what the effect would be if two of the floods (which were actually about 1.5 times the average) had been somewhat greater. With this slight change the curves would be practically identical with the curves for eastern rivers, as shown on Fig. 1. In other words, the data for the Connecticut indicate a difference in frequency relation from the average of all eastern rivers only in that two floods are smaller than the normal. If any two of the seven or eight largest floods had happened to be greater by from 30 to 40%, the relation would have agreed with the general one. That such floods will occur on the Connecticut, and that, in another 70-year period, the data may indicate a relation equal to or exceeding the normal one, is probable. There are no continuous records for any streams in America of a length greater than that on the Connecticut, and the writer submits that curves, based on single records like those given in the discussions, do not justify any change of the relation from the average, unless other evidence is produced.

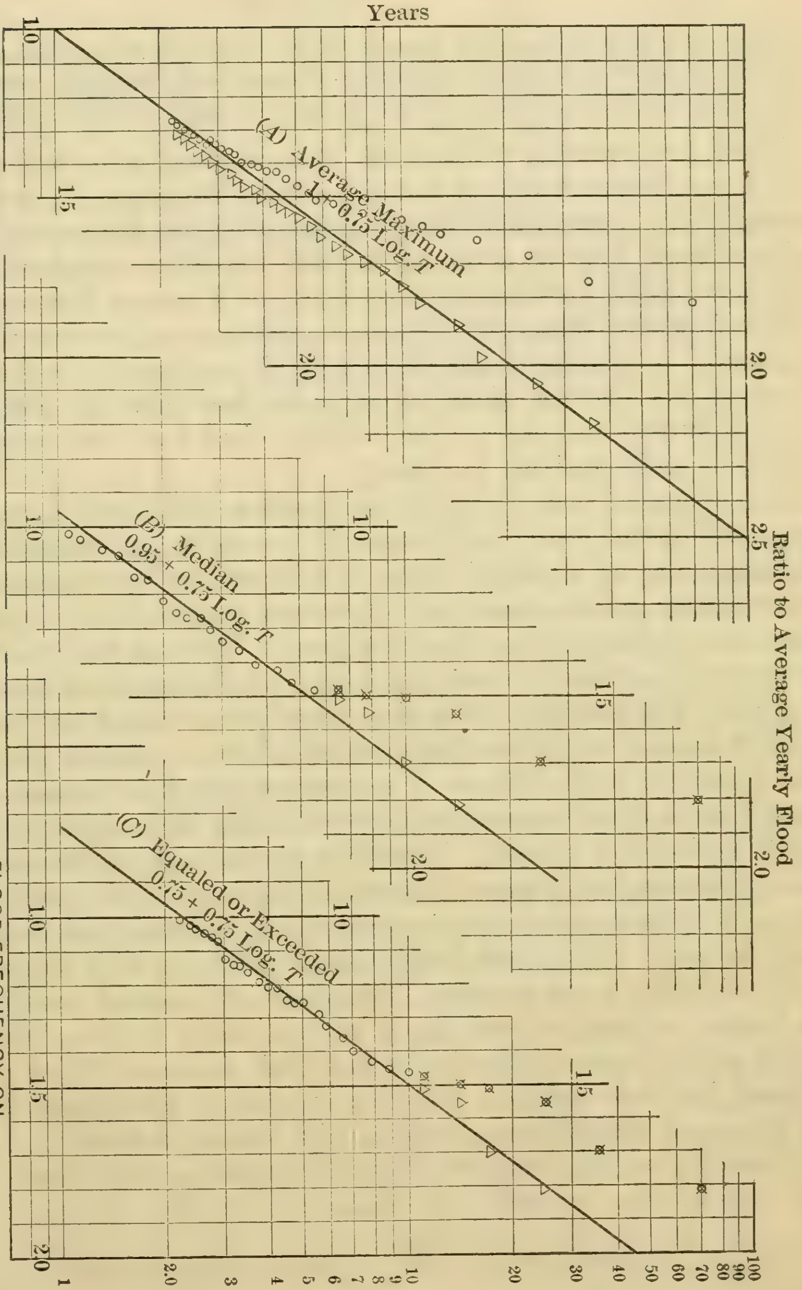
Comparison of Proposed Formula with Others.—Mr. Kuichling suggests a new formula, for use in the South Atlantic States, similar in form to his other well-known formulas. This and other formulas derived in a similar way give relatively higher values for small streams than the writer's formula. There is an essential difference in the meaning of these formulas which should be understood. The writer's formula gives the flood which will probably occur on the particular stream in question in a given interval of time. Mr. Kuichling's and other formulas, derived by plotting the maximum floods which have occurred on streams of different sizes, give the greatest flood which has occurred on any of the large number of streams in the varied intervals covered by the several records.

As there are many more small streams than large ones, it is obvious that there are more chances of obtaining an extraordinary flood on some one of the many small streams than there are of obtaining a similar flood on one of the few large ones.

As an illustration, take Mr. Kuichling's formula for the rivers of the South Atlantic States, which he states is

"based on the greatest observed discharges of the Potomac River at Point of Rocks, Md., the New River at Radford, Va., the Catawba River at Rock Hill, N. C., the Little Tennessee River at Judson, N. C., Cane Creek at Bakersville, N. C., and numerous other streams which

Mr.
Fuller.



⊠ Position of Points if Increase is not made
△ New Position of Points showing Effect of Increase
○ Points not affected by Increase
Average of 5th and 6th flood
increased 40%

FLOOD FREQUENCY ON
CONNECTICUT RIVER
SHOWING
EFFECT OF INCREASING TWO FLOODS

FIG. 18.

Mr. Fuller. exhibit somewhat smaller rates of discharge than the preceding. This new formula is

$$q_{max.} = \frac{41.6\ (620 + M)}{24 + M} \qquad \text{(3)}$$

in cubic feet per second per square mile, and it may be regarded as applicable to mountainous and hilly water-sheds having areas of not more than 10 000 sq. miles, in the portion of the country indicated."

There are probably fully 1 000 streams similar in size to that of Cane Creek for each one of the size of the Potomac. It is then to be expected that there will be many times as many chances of obtaining a single great flood on some one stream like Cane Creek as there will be of obtaining a flood relatively as great on a stream like the Potomac.

The formulas thus derived have an unbalanced element which must be taken into account. Mr. Kuichling has allowed for this to some extent, as his formula gives values much less than the recorded flood on Cane Creek, but the writer believes that the value given by the formula is still relatively too high.

The recorded size of floods on streams like Cane Creek, Devil's Creek, and others which are greatly in excess of any well-verified flood discharges are of doubtful value. A study of the methods used in gauging such floods shows that the measurements are obtained from slopes and sections taken after the flood. Little is known of the conditions which existed during the flood, and the apparent slope is often in error. High-water marks may have been caused by the backing up of the water by obstructions which afterward passed on under the flood pressures. The failure of structures may have caused great discharges for a short interval, which would not have occurred if the obstructions had not existed. The effect on the rate of flow of a stream due to the carrying of great quantities of débris which catch on fences, trees, and other obstacles causing eddies and reducing the channel area, is not known. If one may judge from experience with obstructions in pipes, this effect must be large. The effect of washing away the banks, the failure of bridges and dams, and of other matters which occur during such disastrous floods, is but little known. Flood discharges obtained by such methods are often not even approximately correct, and too much dependence should not be placed on them.

It may be well to state here that, in comparing the writer's formulas with others, the formula, $Q\ (\text{max.}) = C\ A^{0.8}\ (1 + 0.8 \log. T) \left(1 + \frac{2}{A^{0.3}}\right)$, should be used, as in most cases at least the maximum rate of flow is given by the other formulas.

Selecting a Value for C.—The selection of the proper value of C is important in the use of the proposed formula. Before its selection, a study of the flood data on the stream in question and on other

Mr. Fuller.

streams of similar nature in the vicinity should be made. This study should be just as complete and thorough as it would be if any other method of determining the maximum flood were to be used. The use of the average flood facilitates the determination where means are at hand to determine it. The values of C in Column 8 of Tables 12 to 26, inclusive, were obtained from published records. These values are useful for comparison, but it should be understood that the accuracy of the measurements of the floods from which they were obtained should be verified before use is made of them. In any event, coefficients are directly applicable only at the point where the measurements were made, or at other places where the conditions are similar.

Before selecting a value for C from the average flood, any changes in the river itself or in its catchment area which would affect the value should be studied. Additional storage, congestion of the channel, and other matters may change the flood-producing capacity of a stream, and a value of C taken from past floods may not always apply to present conditions.

For rivers on which only short-term records are available, the average flood may be much in error. In order to ascertain the probable accuracy of the average flood, as obtained from records of different lengths, a study has been made of the accuracy of the averages obtained from shorter records on those rivers where long records are available. In this study, use was made of the probability paper devised by Mr. Allen Hazen, and the general method followed was similar to that used by Mr. Hazen in a paper recently presented to the Society.* Details of this method will not be necessary, and only a brief discussion of the method will be given.

For use in the study, records of 15 years or more were utilized. It was assumed that the average of the total records is the true average. The long records were then divided into a number of shorter records, and the average flood, as indicated by these shorter records, was obtained. The ratio of these average floods to the assumed true average flood was found. The ratios thus obtained from all the records of different rivers were combined, on the assumption that the chances for errors on all streams were equal. Plottings were then made of these ratios on probability paper, as shown on Fig. 19. From these plottings Table 44 was prepared, the errors, for 20- and 25-year records being found by extending the curve plotted for 5-, 10-, and 15-year records.

In cases where the value of the coefficient, C, is below that of other streams in the vicinity, or where other evidence is at hand to indicate that the coefficient, C, is too low, an increase in the value of C by at least the probable error would be justified. A further increase

* "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply," *Proceedings*, Am. Soc. C. E., November, 1913.

Mr. Fuller. to provide a factor of safety depends largely on the importance of the works under consideration, and on how great a factor of safety has otherwise been provided for in the selection of the value of *T*.

TABLE 44.

Length of the record, in years.	Probable error. The chances are 1 in 4 that the true average will exceed the average obtained by the record by the percentage given below.	The chances are 1 in 10 that the true average will exceed the average obtained from the record by the percentage given below.
5	11½	22
10	7.5	14
15	5.5	10
20	4.0	7½
25	3.5	6½

Selection of the Value of T.—In the paper the writer has given his views on the method of selecting the value of *T*. He wishes, however, to call attention to the necessity of using a large value for all important work. The use of a large value of *T* is much the same as the use of a factor of safety in other engineering works. If we should build structures on a thousand different rivers, using a value of *T* equal to 1 000, we should do so on the expectation that in each 10 years some five of these structures would be called on to stand a flood in excess of that provided for. For large and important structures, the failure of which would be disastrous, a larger value of *T* than 1 000 should be used. Where the cost is not prohibitive, providing for floods of from four to five times the average yearly flood does not seem unreasonable for such works.

Mr. Morgan suggests that during some years storms occur which cause great floods on many different rivers, and that this may influence the frequency relation. Undoubtedly more great floods do occur in some years than in others. If the study were based on floods on streams of similar size in one section of the country, the effect of these periodic storms might be great. The writer does not believe that the effect of such periodic storms has materially influenced the proposed general frequency relation, for the following reasons: In deriving the relation, streams of greatly different size are utilized. Great floods on small streams are due to very heavy local storms which do not necessarily affect the largest rivers. The streams considered are widely distributed. During each year, on some streams, floods occur greater than any previously recorded. Some of the records used in establishing the relation go back from 50 to 100 years, and the records are pretty well distributed over the last 20 years. The ten greatest relative floods, as given in Table 9, occurred in eight different years. A study of Columns 7 and 8 of Tables 12 to 26, inclusive, shows no difference in frequency between streams with long and with short records.

Mr. Fuller.

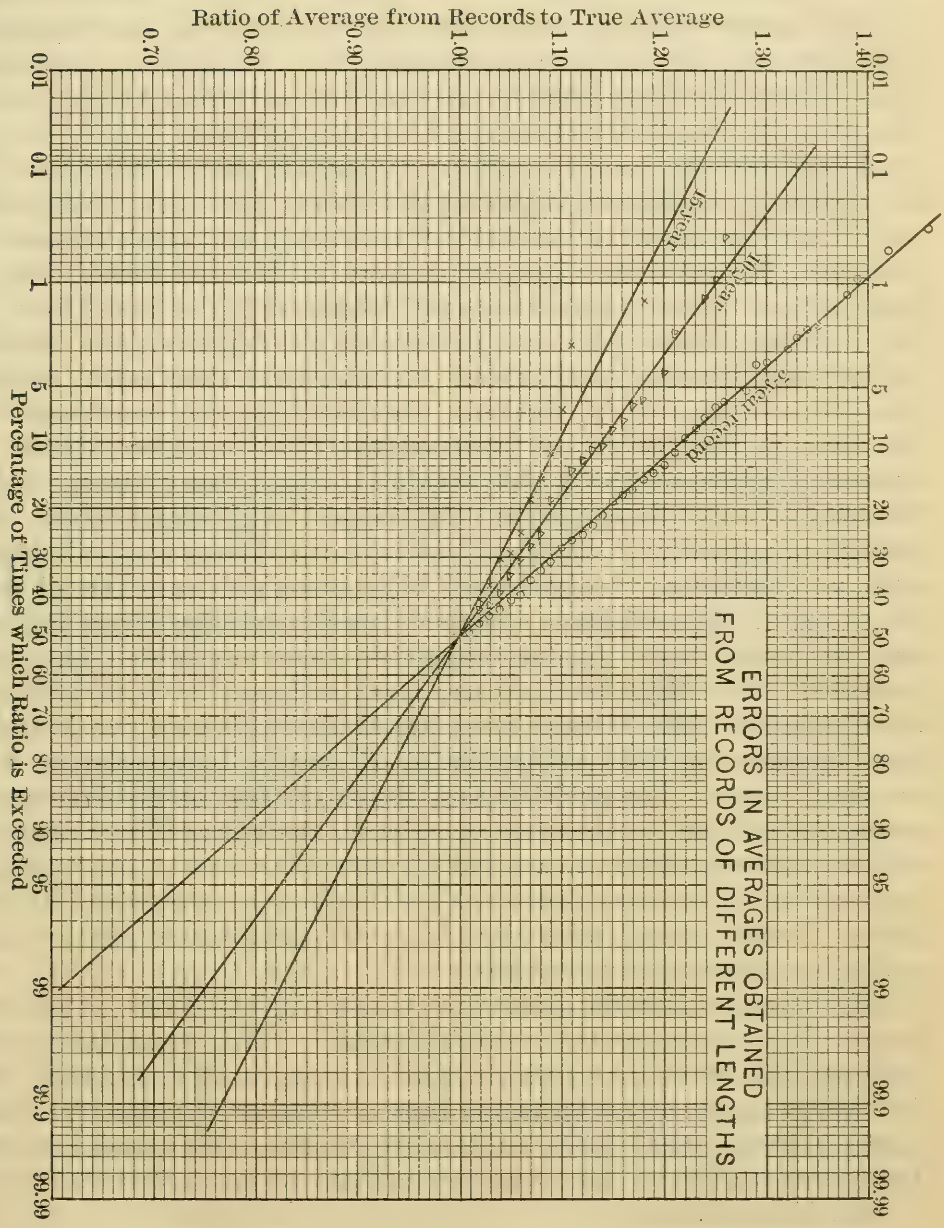


Fig. 19.

Mr. Fuller.

Mr. Morgan states:

"On large water-sheds of 10 000 sq. miles or more, excessive rainfall in one part of the water-shed is usually balanced by lack of rainfall in another part, and the ratio between the average annual flood and the maximum possible flood must be less than for small areas."

The writer does not agree with Mr. Morgan in this statement. The difference in the average intensity of rainfall over catchment areas of different sizes occurs yearly as well as during longer periods, and does not necessarily affect the frequency relation.

A study of Tables 12 to 26, inclusive, shows no indication that the frequency relation is greater for small than for large rivers. It will also be noted that in Table 9 both large and small streams are included. Mr. Morgan cites, in support of his statement, that the large alluvial rivers along the lower Mississippi, the Red River and the Arkansas River, show no indications of deposits from previous great floods, though small rivers elsewhere do show such deposits. Mr. Morgan apparently overlooks the fact that the construction of the levee system, which has confined the floods on these rivers, and prevented much of the great overflow or temporary storage which formerly occurred, has increased the stages during recent floods. Still higher stages will occur on these rivers, as the levees are built higher, until such times as the levees are high enough to care for all floods which occur. The existence of gravel deposits on other rivers indicates past floods of much greater magnitude than any which have been recorded. Such deposits, however, may be found on both large and small streams. Mr. Morgan cites the floods on Devil's Creek and on small streams in Ohio as indicating extremely large relative floods on small rivers. For such streams there are few or no data on the average flood, and, in the writer's opinion, the recorded measurements of the maximum flood are not sufficiently reliable to be used for such comparison. The writer believes that the flood on the Miami at Dayton—a river of considerable size—is one of the greatest relative floods which has occurred in recent years.

Mr. Morgan uses the term "maximum possible flood" as applying to the flood given by the writer's formula. This formula gives no limit to the maximum possible flood, but gives the probable flood. Mr. Morgan calls attention to the limited number of years represented in the table of great floods on foreign streams. This does not affect the formula, as these data were not utilized in its derivation.

Mr. Morgan states:

"It might be better to estimate the maximum possible flood by what might be called the rational method, that is, by determining from the basis of experience and the maximum rainfall to be expected, the relation of rainfall to run-off under the conditions which would exist in an assumed case, considering the elements of topography, shape of the drainage basin, direction of storms, season of the year, etc."

Mr. Fuller.

There is quite as much uncertainty in probable maximum rainfall as in probable flood flow. There is, in addition, great uncertainty as to the percentage of run-off to be expected on different rivers, and under different conditions. We know but little of the effect of topography, direction of storm, shape of catchment area, and other factors. To the writer there seems to be much less chance for error in selecting the value of C from the average yearly flood, or by a study of the values of C for other rivers, than in estimating values for these many unknown quantities.

Many of the important points brought out in Mr. Morgan's interesting and instructive contribution have been covered in the writer's general discussion, and need no further comment.

Mr. Hinckley's interesting remarks, made in 1911, indicate clearly how engineers have looked at the probability of the occurrence of floods. His rainfall data are interesting. The application of probability methods for obtaining the probable rainfall would undoubtedly give much valuable information.

Mr. Chandler presents some valuable data for the flow of the Red River of the North and its branches, and draws some interesting conclusions as to the frequency relation of streams in that section. He says that the conditions in the Red River Valley are similar to those in a partly arid region, so that the writer's average relation does not hold. Although the data available are not sufficient to be conclusive, Mr. Chandler's suggested frequency relation gives an indication of how much higher it may be in such sections. The size of the average flood for such rivers is governed largely by the number of dry years in the period. It occurs to the writer that in determining the average flood, the exclusion from the record of all years so dry that no real flood occurs, may have merits, thus giving an average of the real floods as a basis of comparison. On this basis the relation may be expected to be more like that for other sections.

The writer notes that Mr. Chandler very properly uses the median flood method in his study. This method is better for plotting short records, as the few largest floods do not affect the plotting of the other floods to the same extent as in the average maximum flood method, and by its use a closer approximation to the true curve is obtained.

Mr. Hazen points out clearly the uses and limitations of the proposed formulas. The map of the country (Fig. 7) is most useful, as it indicates the data available for different sections of the country and also shows the variation in the value of C for rivers in the same section. A study of the conditions will in most cases account for such variations. For instance, in Maine, where the coefficients vary from 17 to 110, it will be found that the low coefficients are for small streams draining extensive systems of lakes which control largely the floods, and the large values are for streams on which there is little or no storage.

Mr. Fuller.

It is interesting to study the effect of this storage on the river system. For the upper branches, the effect is great, but, as the streams join to form the larger rivers, the effect decreases rapidly until, near the mouth of the river, the coefficients are in some cases several times as great as for the upper branches. The study of streams in that State, where there are such extensive lake systems, is most illuminating in regard to storage for flood protection.

As Mr. Hazen states, probability paper might have been utilized in the study of flood frequency. During the study for this paper the normal probability curve was tried, but it was found that the data did not fit it as closely as the logarithmic curve adopted. The use of probability paper, which provides a ready means of drawing curves varying somewhat from the normal law, allows this method to be used. Practically, however, such a curve would give results identical with the formula, within the limits of the data, and an extension of the curve would indicate a ratio of 4.0 for $T = 10\,000$, as compared with 4.2 by the formula. Such differences are of little moment, and no data are available to indicate which is the more nearly correct.

The method of plotting gauge heights on probability paper, as suggested by Mr. Hazen, should prove useful. The writer has plotted records of the gauge heights for many stations along the Mississippi and its branches. Although these plottings are not in all cases as close to a straight line as the one for Cincinnati, they all approximate such a line. Within such limitations as may be applied by one having a thorough knowledge of the river, both as to changed conditions affecting the stages and as to the storage and increased channel capacity which occur at higher stages, this method seems applicable to many rivers.

Mr. Hazen's suggestion in regard to determining coefficients of variation, as an index of how closely the stream follows the average law of frequency, is interesting. It may be that a thorough study of the regularity of flow throughout the year, and of other characteristics of the stream, would enable the effect of some factors on flood frequency to be ascertained.

Mr. Knowles, citing a record of a single stream as an indication of variation from the average, warns against the general use of the average relation. The writer pointed out in the paper, and has further stated in this discussion, that a thorough study of all local conditions should be made before using the formula, but must repeat that, in his opinion, the data on a single stream are entirely inadequate for establishing a frequency relation. It could be shown by a plotting similar to Fig. 18 that the data for the Allegheny River at Kittanning really differ but slightly from the average relation. Mr. Knowles' extension of the existing record to a 41-year record, by estimating the flood at Kittanning from the gauge height at other points on the river

is of doubtful accuracy. An examination of Figs. 10 and 11 will show that the actual floods which Mr. Knowles plotted in order to obtain the curves showing the relation between the gauge heights at Kittanning and those at Freeport and Parker differ, in some instances, by from 50 to 100% from the curves. If these actual floods do not agree more closely with the curves, it is obvious that similar errors probably exist in the floods obtained in this way and included in the record. As the effect of the few large floods is so important, it is clear that a record obtained by such methods should not be used in discussing frequency. Mr. Fuller.

Mr. Bellamy's discussion is interesting, and gives much valuable information as to flood flows in Australia. The large variation in floods for the different rivers is to be expected in a country so large and with such widely varying conditions. As Mr. Bellamy states, a comparison of such rivers, with the limited data available, is very unsatisfactory. It is much the same as comparing rivers of the American coast, where the coefficients are in many cases greater than 100, with rivers in the Missouri River Basin or the Great Basin, where the coefficients are in many cases less than 10.

Mr. Kuichling has supplied tabulated data relating to great floods which have occurred on both foreign and American rivers, and these, taken in connection with those previously presented by him, give by far the most complete record of maximum flood flows available. The data from which this information was prepared are widely scattered. In many cases, particularly for foreign streams, the original data are in such a state that most careful study of them is required before they can be used. The collection and analysis of this large mass of data should be appreciated by all who are interested in the flood problem.

Mr. Kuichling brings out clearly many important aspects in regard to flood flows. Some of these points have been taken up by the writer in the preceding general discussion, and need no further comment.

Mr. Kuichling calls attention to the wide variation in the value of C for different rivers in the same general section of the country, and suggests that it may be better to use a single value of C for one section. Where coefficients are based on long records, the writer believes that the differences in the values of C indicate different flood-producing capacities for the streams. In many cases these differences may be accounted for by a study of the catchment area, as to slopes, storage capacity in lakes, and rainfall conditions. Where records of considerable length are not available, the writer agrees with Mr. Kuichling that much consideration should be given to the coefficients for other streams in the section. He is correct, in his interpretation of Tables 12 to 26, inclusive, in stating that the values of C in Column 8 were the ones intended for general use. The values in Column 8 are derived from the average flood, and take into account all the largest yearly floods in the rivers; they are more accurate than those in Col-

Mr. Fuller.

umn 7. It would have been better, as he suggests, to have called these C_1 and C_2.

Mr. Kuichling calls attention to the important work of Iszkowski, and reduces his formula to conditions for the United States. Although this formula is of very different form from the writer's, it may be of interest to show how the results obtained from it agree with those obtained from the writer's formula,

$$Q\ (\text{max.}) = C\,A^{0.8}\,(1 + 0.8 \log. T)\left(1 + \frac{2}{A^{0.3}}\right).$$

Under what may be regarded as similar conditions, Iszkowski's formula differs from the writer's by the following percentages: for a catchment area of 10 sq. miles, — 21%; 100 sq. miles, + 32%; 1 000 sq. miles, — 2%; 10 000 sq. miles, — 39 per cent. This comparison is made on the basis of the values deduced by Mr. Kuichling from Iszkowski's formula for "hilly country, slightly permeable soil, and sparse vegetation" and of a use of $C = 100$ and $T = 100$ in the writer's formula.

The writer has plotted the largest of the floods in America, as given by Mr. Kuichling, on the lower diagram of Plate LXIX, and finds that with the exception of three or four extreme floods, such as those on Cane Creek and Devil's Creek, the plottings agree with the curves representing the writer's formula as well as the points previously plotted.

Mr. Horton calls attention to a discussion of the use of probability methods in a report made by Mr. Rafter in 1896. This discussion is interesting and instructive. That paper, however, is entirely confined to a discussion of rainfall and minimum run-off. The writer fails to find any suggestion as to the use of similar methods for determining flood flows. Until Mr. Horton called his attention to it, the writer was not aware of the existence of this study. Mr. Horton further states that Mr. Rafter suggested to him the use of probability methods for determining the probable flood as early as 1896 and that he has used it on numerous occasions in his professional work. The writer had no means of knowing about any work which Mr. Horton had done on this subject, and sees no reason why he should qualify the statement that the original suggestion came to him from Mr. Hazen.

Mr. Horton gives three different formulas derived from the data on rivers near Philadelphia. He also gives the following general formula for which he states that he

"does not claim any great breadth of applicability for the general formula, * * * but believes that factors other than area modify the flood discharge of streams so profoundly that it is better, wherever possible, to derive individual formulas or flood-frequency diagrams for each stream."

This formula is: Mr. Fuller.

$$Q = 4021.5 \frac{T^{\frac{1}{4}}}{A}$$

in which Q is the flow in cubic feet per second per square mile. To put this formula on the basis of the writer's, that is, to give the total flood flow of the stream, it becomes necessary to multiply by A, so that the formula becomes

$$Q = 4021.5\ T^{\frac{1}{4}}$$

According to this formula, the maximum floods to be expected are independent of the catchment area. Surely a formula which does not take into account the size of the catchment area can have no general application.

During the early part of the study for this paper, exponential formulas such as those proposed by Mr. Horton were tried. It was found that though these gave curves closely following the data for short periods, for the longer records the exponential relation was less satisfactory than the logarithmic relation adopted.

The writer deems it unfortunate that, after seventeen or more years of consideration of the use of probability methods for the determination of flood flows, Mr. Horton should have confined his discussion so largely to criticizing details of the writer's methods, instead of giving more in regard to his own studies.

Mr. Horton states:

"It would appear that the relation between flood magnitude and frequency, when expressed in terms of either 'average maximum flood' or 'median flood,' is not only very indefinite and difficult of comprehension, but is apt to be very misleading, if it is not indeed practically meaningless. It does not directly convey the information which the engineer usually most desires, for example: If a spillway has a capacity of 1 000 cu. ft. per sec., how often on an average will its capacity be exceeded?"

In regard to this statement, the writer will call attention to the fact that the others who have discussed the paper have not found the proposed formula either indefinite or misleading, but, on the contrary, have comprehended its true meaning. The writer, on pages 1022-1029,* described three different methods which are applicable to flood frequency problems. In Table 6 the formulas representing these methods are stated, and values are given for the volume of the flood indicated by them in different periods of time. If any one wishes to know the volume of the flood that will probably be equalled or exceeded in a period of time, he may do so by using the formula:

$$Q \text{ (equalled or exceeded)} = C\,A^{0.8}\,(0.7 + 0.8 \log.\ T)$$

* *Proceedings*, Am. Soc. C. E., May, 1913.

Mr. Fuller. as given in Table 6. There are problems in which it is useful to know the volume of this flood, but the writer believes that the most important question which the engineer desires answered is: What is the probable flood for which to design our structures? For this purpose we surely do not wish to ascertain the smallest of the maximum floods that is likely to occur in the given interval, which is only a different way of stating the "flood to be equalled or exceeded." For example, suppose we have a record which includes ten periods, each of 20 years. Each of these periods would have a maximum flood of a different size. The flood to be equalled or exceeded in a 20-year period would be the smallest of these. To design works which would just provide for this flood would mean that in all probability the design would prove inadequate within the period, because during nine out of ten periods, greater floods would occur.

The average of the ten maximum floods which have occurred in the several periods, as given by the writer's formula, seems to be a logical basis for design in order to provide for a reasonable chance that the structure will survive the given interval. The median of these floods, or the one for which there is one greater for each one less, represents the flood for which the chances are even that it will occur in the given period. The use of this flood is as logical as the use of the average. The difference in value between the two is slight, and the writer, for the reasons stated in the paper, prefers the average maximum.

Mr. Horton, after giving a method of determining the "average interval of recurrence of a flood lying between any two given magnitudes", states: "This illustrates what seems to be a fatal error in the author's method of analysis."

The writer fails to find in the paper any statement which would justify the interpretation Mr. Horton has given the formula. As the method of derivation and the meaning of the formulas are explained in considerable detail in the paper, the writer considers that to apply it as Mr. Horton has done is entirely unwarranted. If the determination of the average interval of recurrence of floods of a size between fixed limits is desirable, as it may be for some special problem, the use of the flood to be equalled or exceeded, as suggested by Mr. Horton, is proper. Such use, however, is an unusual one, and to find a "fatal error" in the average maximum flood formula because it is not applicable to such a problem, is like condemning a formula proposed for determining the strength of a steel beam because it does not give the deflection of the beam. The writer was aware that the "flood to be equalled or exceeded" and the "median flood" had their uses, and accordingly gave the formula by which they might be obtained and descriptions of the methods of derivation, but, to

explain their application to special problems, he thinks is beyond the scope of the paper. Mr. Fuller.

Mr. Horton considers the use of ratios "confusing and cumbersome". By the use of ratios it becomes possible to compare the frequency of occurrence of floods on different rivers, which, in the writer's opinion, is of the greatest importance.

Mr. Horton objects to the writer's method in that he does not use all the floods to determine the frequency relation. As a matter of fact, in the determination of the general formula, 1 672 floods were considered; 20 of the largest were plotted individually and 200 others in groups of ten. This plotting covered three-quarters of the total length of the curve, as plotted in Fig. 1. It is apparent that, with the average maximum method, the average flood, which has a ratio of unity, must be plotted for 1 year. A straight line drawn from unity through the average of the points plotted satisfied the requirements, thus making the plotting of the remaining points so obviously unnecessary that it was not done. The writer assures Mr. Horton that the remaining points will fall close to the line. Mr. Horton's other criticisms of the paper are, the writer believes, fully covered by the foregoing discussion.

Mr. Pillsbury's discussion is an interesting and natural one. An examination of Fig. 18 shows how a slight change in the data on the Connecticut River would affect the frequency relation, and the writer has already given his views on the reliability of frequency relations established from the records of single streams. Aside from this, the assumption made by Mr. Pillsbury contains a fundamental error which would have to be, and could be, eliminated. This error is in assuming that the series of annual floods follows the normal law of probability. By the normal law of probability, the variations upward and downward from the normal are equal. As a matter of fact, with flood data, the variations upward and downward are not equal. The variations downward are more numerous, but the variations upward are greater in magnitude. The values follow, not the curve of normal probabilities, but what is called a skew curve. If Mr. Pillsbury will deduce the formula that most accurately represents this skew curve, and will show its application, he will give a useful solution of the problem which he has attempted, and will be doing the Profession a real service.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

CONCRETE BRIDGES: SOME IMPORTANT FEATURES IN THEIR DESIGN.

Discussion.*

By Messrs. W. D. Maxwell and Walter M. Smith, Sr.†

W. D. Maxwell, Assoc. M. Am. Soc. C. E. (by letter).—The question of temperature changes in arches has been mentioned by several who have discussed this paper. Mr. Maxwell.

When the Walnut Street Concrete Bridge, of which the writer was one of the designers, was built in Des Moines, Iowa, in 1911, nine electrical resistance thermometers were embedded at various points in one of the arches. This arch has a clear span of 68 ft., a crown thickness of 16 in., and a solid spandrel fill. The greatest range in temperature was at the crown, where an average of two thermometers showed a temperature of 86° Fahr. about 16 hours after pouring, 9° below zero as the low point during the following winter, and 86° in the summer of 1912, one year after the bridge was built. Other thermometers placed in thicker portions of the arch showed somewhat less susceptibility to atmospheric changes. The average of two instruments embedded about 2 ft. in heavy concrete near the pier and abutment showed 98° Fahr. about 36 hours after pouring, 20° the next winter, and 85° the following summer. The average of all the thermometers for the first 6 months, gave a range of temperature in the concrete of 87.6 degrees. The upper limit of this range was the extreme produced by the heating of the setting concrete. The average of all the thermometers for the second 6 months was 79.2 degrees.

The extremes of atmospheric temperature in the summer of 1911 and the following winter were very great for this locality, the total

* Continued from February, 1914, *Proceedings*.

† Author's closure.

Mr. Maxwell. range being 135 degrees. However, ranges of considerably more than 100° may be expected every year.

Elevations were taken on all six arches of the bridge in September, 1911, when the average of all the thermometers was 75°, and again in January, 1912, when the average was 11 degrees. The difference corresponded closely to that computed for a change of 64° in the temperature of the concrete.

In the design of the bridge, allowance was made for a 40° rise, and a 40° fall. These amounts were considered excessive by some engineers, but the actual observations show that they were not only not too small, but that the fall was more than 40°, the concrete having been poured at a general average of about 70 degrees.

Considering these temperature ranges, it is apparent that concrete bridges built in these rigorous climates, especially the hingeless arches, must be very carefully reinforced against temperature stresses, and it is also evident that the warmer climates and those having nearly uniform yearly temperatures possess a real advantage with this type of construction.

Mr. Smith. WALTER M. SMITH, SR., M. AM. SOC. C. E. (by letter).—Mr. Gregory states that the decision to adopt a three-hinged arch for the Traver Hollow Bridge was taken because the foundations for the abutments and piers would be on compressible material. He also states that the preliminary estimates indicated that the three-hinged arch would be somewhat more expensive than the fixed-arch type. He is correct in the first statement, but is mistaken in the second. Only a rough estimate was made for a fixed arch at this site, making certain assumptions as to the depth to rock. Careful designs and estimates were then made for a three-hinged arch, to compare with the other. The writer had charge of both estimates, and made the design and estimate for the three-hinged arch. He has copies of these estimates before him as he writes. If the same unit prices are taken for the three-hinged arch as for the fixed arch, the saving would be about 30% for the former. Therefore the average increase in unit price for the three-hinged arch would have to be 43% in order that it might cost as much as the fixed arch for this site. When these estimates were obtained, as there was no information at hand as to the depth to rock, it was decided to build the three-hinged arch, as it was certain that the material would carry the load of an arch of this type by compressing slightly, and would not injure the arch.

Figs. 10, 11, and 12 show the details of the hinges and the method of connecting them to the steel ribs of the arch. It will be seen that the steel forms a continuous structure between the abutment hinges, with an additional hinge at the crown.

When the design of the Rye Outlet Bridge was under consideration, the writer also investigated a three-hinged arch for this site, at Mr.

Mr. Smith.

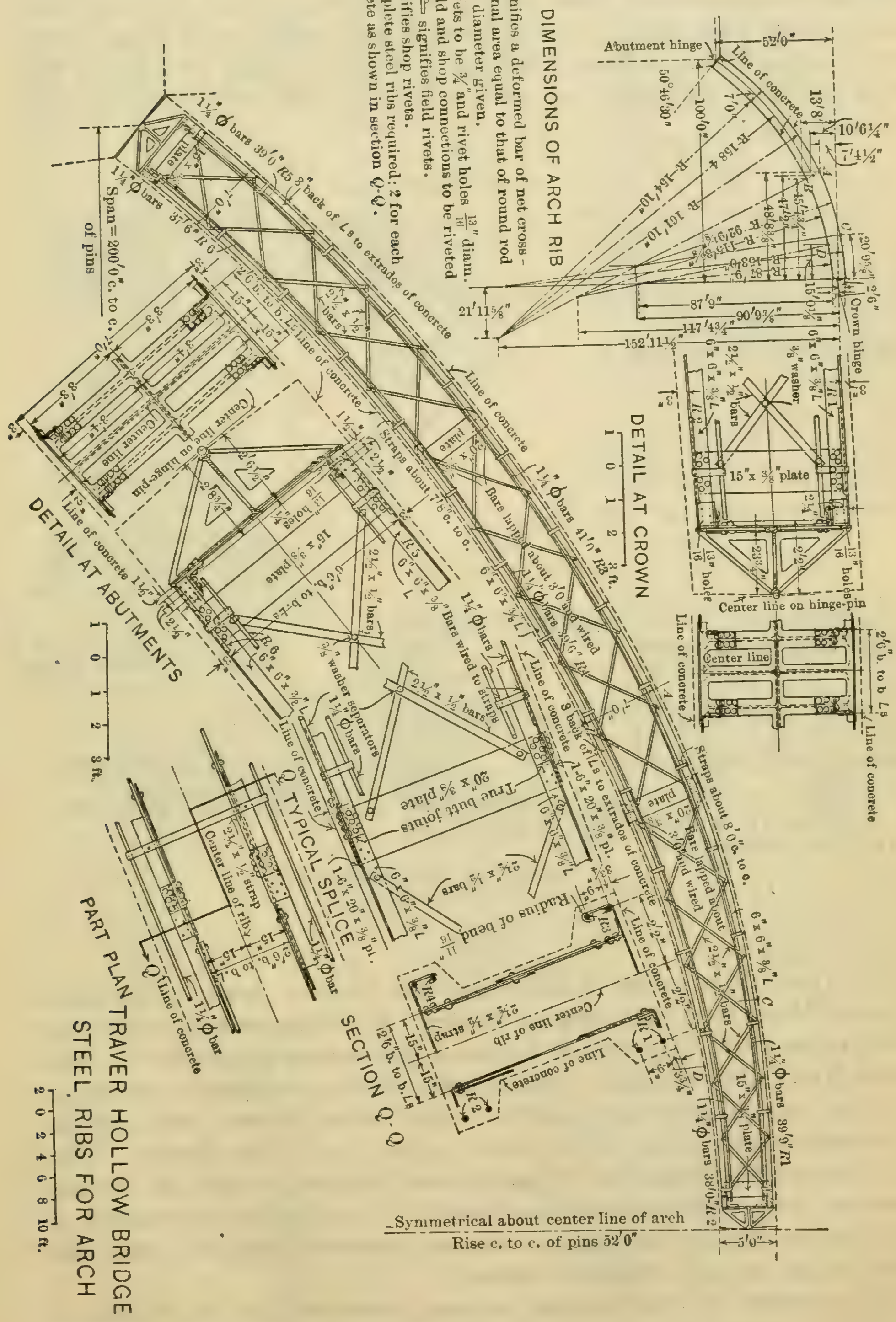


Fig. 10.

Mr. Smith. Gregory's request. As the height of the roadway was fixed, and also the water line, the design was limited to short arches, unless it was admissible to construct them with a ratio of rise to span of more than six, on very high piers. It was thought inadvisable to do this. With short arches the three-hinged type showed such a slight saving over the fixed type that the latter was adopted, as the piers were to be founded on bed-rock.

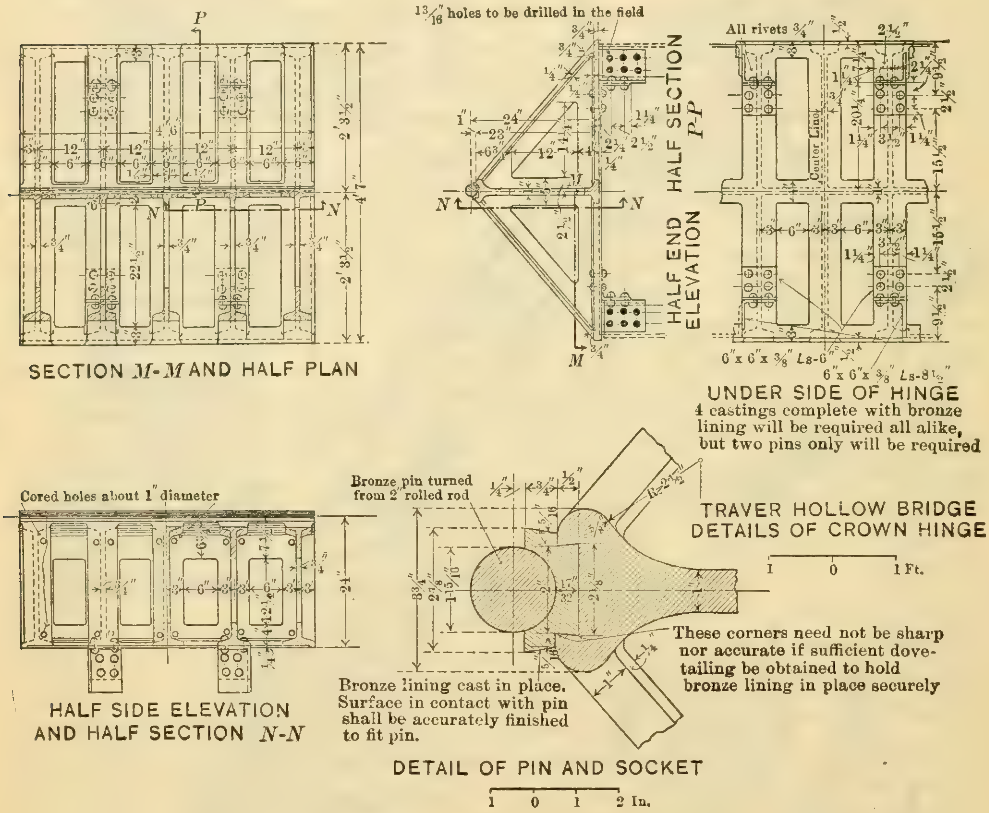
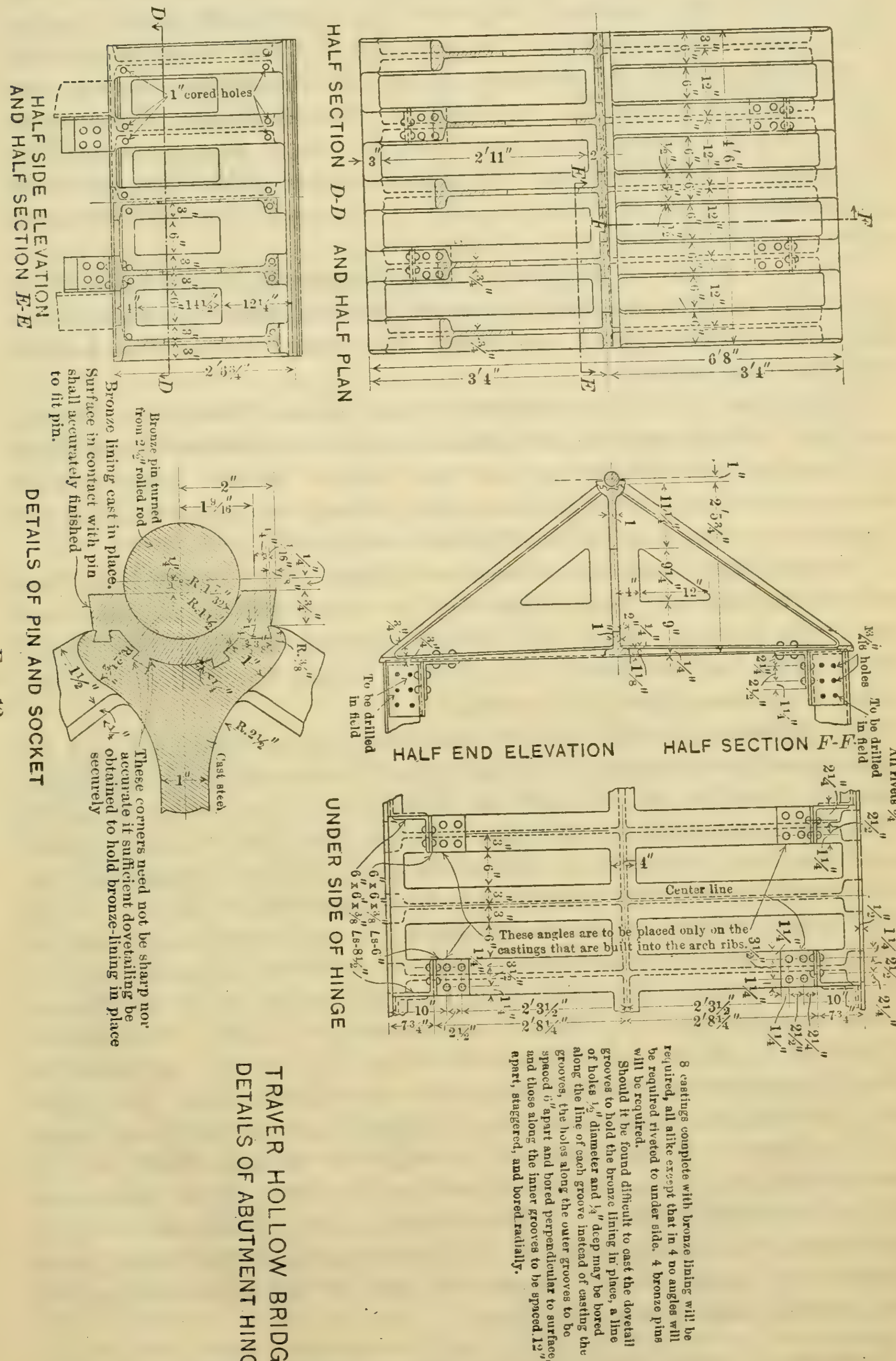


Fig. 11.

Mr. Gregory is wrong in supposing that the pier shown in the paper is one of those in the Rye Outlet Bridge. In reference to this pier, the paper states that, "The first is a braced pier somewhat similar to those in the Rye Outlet Bridge." The thrusts, shown as acting at the top of this pier, are assumed, and, so far as the writer knows, they may not be anywhere near those of the piers in the Rye Outlet Bridge. They were taken arbitrarily, and the piers were designed to suit the thrusts; therefore the writer fails to see how the investigation mentioned by Mr. Gregory has any bearing on this case, however valuable it may have been for the purpose which he states.

The method of constructing an arch, described by Mr. Wilson F. Smith, is a very satisfactory one, if the arch is to be of plain concrete,

Mr. Smith.



TRAVER HOLLOW BRIDGE
DETAILS OF ABUTMENT HINGE

FIG. 12.

Mr. Smith. but, with all due respect to Mr. Janni, who states that he has constructed reinforced concrete arches in this manner, the writer cannot believe that it is economical. The difficulty of fitting the great number of bulkheads around the steelwork is very great, and the expense very heavy. In Pittsburgh, Pa., a reinforced concrete bridge, with a main arch of 216 ft., consisting of two separate ribs, is now being built by this method. With slight modifications, the same method has also been used extensively in America, and also in Germany, for plain concrete arches, and has proved very satisfactory.

The main arches of the Walnut Lane Bridge, in Philadelphia, Pa.,* were built in sections, leaving spaces between many of them to be filled with concrete later, as keys. Mr. Janni states that he has used the same method.

The arches of the Connecticut Avenue Bridge, Washington, D. C., were also built in sections, the spaces between the sections being filled in afterward, but this bridge and also that at Walnut Lane were of plain concrete, and in neither case was a temporary hinge made at the abutment or crown, as in Mr. Morison's method.

On the other hand, many masonry bridges in Germany have been built with temporary stone hinges at the crown and the abutments to insure the passage of the equilibrium polygon through these points for the dead load, the spaces around these hinges being filled with concrete on the completion of the bridge.

Mr. Quimby and Mr. Mensch state that, in the fixed arch and the three-hinged arch, the thrust at the crown is about the same, and therefore Mr. Quimby reasons that the thickness of the crown should be about the same, seeming to forget the fact that, for a heavily unbalanced live load, the thrust at the crown does not pass through the center, though in the three-hinged arch it is compelled to do so. In the Rye Outlet Bridge, which was designed for very severe conditions—a 20-ton road roller concentrated on one rib—this was especially so. At the crown the thrust was about 5 in. from the center; at the abutments, it was more than 12 in.; thus it was necessary to have a thickness of at least 30 in. at the crown and more than 6 ft. at the abutments in order to keep the equilibrium polygon in the middle third. As a matter of fact the thrust at the crown is slightly less in the three-hinged arch, on account of the difference in weight of the arch itself, but the great difference is caused by the thrust being central in the latter case. In the Rye Outlet Bridge, if three-hinged, only a thickness of 20 in. is needed at the crown and 25 in. at the abutments, as the thrust passes through the center at both places, and that at the abutments is only 25% greater than that at the crown.

* "Walnut Lane Bridge, Philadelphia," by George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXV, p. 423.

Mr. Smith.

The point made by Mr. Quimby, that thickening the arch to keep the equilibrium polygon within the middle third will increase the temperature stresses on account of the added stiffness, is well taken, and this is one of the main objections to the fixed arch. How is this trouble to be remedied? Some engineers do so by assuming much less temperature change than the writer believes to be justifiable, thus letting the concrete crack and cause some kind of a hinge of its own if its strength is exceeded. Others do so by allowing higher unit stresses in the steel and concrete. Apparently, Mr. Quimby prefers the former method, judging from his statement of what he believes to be the maximum temperature variation in concrete.

The cracking of the concrete at the abutment or crown, or both, which occurs frequently in fixed-arch bridges for which a sufficient variation of temperature has not been allowed, will not necessarily endanger the arch. It is merely an attempt, on the part of the concrete, to remedy the fault of the designer, and form hinges of its own, thereby readjusting the stress. These cracks, however, are unsightly, and, when they occur, it is hard to convince the layman that the bridge is still safe.

The writer has stated that, with regard to appearance, the three-hinged arch is at a disadvantage, and, when the question of beauty is of sufficient importance, it would probably not be considered, its advantage being where economy is of first importance.

The maximum range of temperature change in an arch is not necessarily the change from midsummer to midwinter, with the condition of "no stress" lying midway between. Unless the arch is built in the spring or fall, and some method (such as that described by Mr. Janni) is adopted to eliminate the stresses due to setting shrinkage by leaving in the arch key spaces to be filled in after the concrete of the arch has been placed, set, and cooled off, this range will be much higher. If these precautions are not taken, the condition of "no stress" in the arch is the position it occupies at the setting temperature of the concrete, and any variation from this position causes deformation of the arch, and, therefore, stress. The variation will be all one way if the bridge be built in the summer. The steel in the arch has no effect in preventing cracks in the concrete, in fact, it generally causes them to occur more quickly than if it were not used. Many engineers have had this experience in building various structures. In building the retaining walls for the Pennsylvania Railroad terminal, in New York City, it was found to be impossible to make the length of the sections of wall greater than 25 ft. without cracks.* The reinforced concrete floor beams, which carry the concourse floor of vault light construction in the Pennsylvania Railroad Station, New York City, show cracks where they join the columns at almost 50% of the points. These were built as continuous beams over great distances, and the setting shrink-

* *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 334.

Mr. Smith. age caused the cracks. The beams are not necessarily unsafe because of these cracks.

In 1908 a high, heavily reinforced concrete retaining wall was built along the railroad yard on the water-front at St. George, Staten Island. It is several thousand feet long, and was built without expansion joints. In the fall of that year the writer was informed, by one of the most prominent engineers connected with this work, that expansion joints had not been provided as they were not needed, the quantity of steel being sufficient to prevent it from cracking. The writer had seen some of the effects of this before, so in March, 1909, he inspected carefully about 1 000 ft. of this wall, and in that length counted 45 cracks extending from the base to the top, and 17 extending part way, making an average of about one crack in every 16 ft. Through 8 of these cracks water was issuing at various heights.

A similar experience was had in the top of the Cross River Dam. George G. Honness, M. Am. Soc. C. E., Division Engineer in charge of its construction, states that:*

"It was not expected that the use of these rods would entirely eliminate the temperature cracks, but it was hoped that it would prevent their concentration in a few large cracks.

* * * * * * *

"The reinforcement * * *—as to quantity and method of placing—was not effective in preventing the concentration of the cracks."

The following is given as an example of how greatly concrete shrinks in setting. On some work with which the writer was connected, some years ago, it became necessary to cut out some concrete in a ceiling consisting of a very deep and heavy concrete slab with reinforcing bars embedded about 2 or 3 in. from the lower surface. The concrete was cut from around a few of these bars for a distance of about 2 ft. All the bars, when released, sprang out of line, thus showing that the shrinkage of the concrete, in that short distance, was sufficient to shorten the steel bars by compressing them. Theoretically, these bars were carrying a tension of 16 000 lb. per sq. in. In reality, they were in compression, and the concrete was not only carrying all the load, but was also compressing the steel.

Many experiments have been made in order to determine the rise in temperature in cement mortar, neat and of various compositions, and of concrete during setting. In some tests by the Universal Portland Cement Company on neat cement cubes, the average temperature rise was from 80° to a little more than 130° Fahr., in from 6 to 8 hours after placing. In the case of concrete, the initial temperature, 70° Fahr., was increased to 80° in 8 hours, 100° in 25 hours, and 105° in 45 hours. Tests in the laboratory of Lehigh University have shown

* *Transactions*, Am. Soc. C. E., Vol. LXI, pp. 411-413.

that the temperature of an 8-in. cube of 1:3 Portland cement mortar rose from an initial temperature of 70° to 160° Fahr. in 18 hours. At the Watertown Arsenal, a few years ago, tests of 29 cubes of neat cement, representing 27 brands of cement, were made. The initial temperature was 77°, and the average maximum 185° Fahr., showing a rise of 108° in an average of 13 hours. The thermophones embedded in the cyclopean masonry of the Boonton Dam indicated a rise of from 20 to 25° Fahr., although only about two-thirds of the mass consisted of concrete. In the locks of the Panama Canal the rise was also about the same. Mr. Smith.

Table 1 shows the setting temperature in some concrete canal walls at Madison, Me., built for the Hollingsworth and Whitney Company. As the concrete was placed in the winter, it was first heated to 70° Fahr. The average atmospheric temperature was 11° and that of the concrete 108°, showing a rise of 38° after being placed, in spite of the exceedingly low outside temperature.

TABLE 1.—Setting Temperature in Concrete Canal Walls at Madison, Me.

Date. 1909.	Depth of concrete, in inches.	Atmospheric temperature, in degrees, Fahrenheit.	Temperature of concrete, in degrees, Fahrenheit.
Jan. 19th	3.0	— 18	91
20th	3.0	— 2	94
21st	7.5	14	96
22d	15.5	34	98
23d	15.5	39	106
24th	15.5	20	110
25th	15.5	15	111
26th	15.5	18	113
27th	26.25	2	119
28th	35.0	18	119
29th	35.0	14	116
30th	37.0	21	116
31st	37.0	20	113
Feb. 1st	37.0	0	113
2d	37.0	— 5	110
3d	37.0	— 10	109
4th	37.0	— 11	113
5th	37.0	23	114
6th	37.0	28	106
7th	37.0	17	104
8th	37.0	6	101
9th	37.0	— 4	98
Averages		11	108

As an example of the low temperature which may be reached in bridge arches in the Northern States, Table 2, showing the results of tests on the Squaw Creek Bridge, at Ames, Iowa, is interesting.

Mr. Maxwell has given a brief account of the range of temperature experienced in the Walnut Street Bridge, Des Moines, Iowa, and

Mr. Smith. readers are referred to a synopsis of his original report on this bridge, and also on some others.* Many of the data herewith presented have been obtained from Mr. Maxwell's valuable report. Tables 3 and 4, taken from that report, are interesting.

TABLE 2.—RESULTS OF TESTS ON SQUAW CREEK BRIDGE, AMES, IOWA.

Thermometer.	Maximum.	Minimum.	Range.
A1	82.2°	— 2.5°	84.7°
B1	79.5°	2.0°	77.5°
A2	80.5°	8.0°	72.5°
B2	81.4°	— 0.5°	81.9°
Average range			79.1°

TABLE 3.—TOTAL TEMPERATURE RANGE FOR THE WALNUT STREET BRIDGE.

Coil No.	Minimum temperature, in degrees, Fahrenheit.	Maximum temperature, in degrees, Fahrenheit.	Range of temperature, in degrees, Fahrenheit.
PERIOD FROM AUGUST, 1911, TO JANUARY, 1912.			
2	15	108	93
3	20	102	82
4	19	95	76
5	18	96	78
6	0	88	88
7	— 5	90	95
8	— 9	86	95
9	— 9	85	94
Average range			87.6
PERIOD FROM JANUARY TO AUGUST, 1912.			
2	15	84	69
3	20	83	63
4	19	86	67
5	18	85	67
6	0	87	87
7	— 5	86	91
8	— 9	84	93
9	— 9	88	97
Average range			79.3

The coils were in different portions of the structure, and at different distances from the surface. Coils Nos. 2 to 5 were placed in the concrete where the arch joins the abutments or piers. No. 1 in the

* *Engineering and Contracting*, November 12th, 1913.

Mr. Smith.

TABLE 4.—Rise in Temperature Due to Setting.

Coil No.	Initial temperature of concrete, in degrees, Fahrenheit.	Maximum temperature of concrete, in degrees, Fahrenheit.	No. of hours after placing.	Rise of temperature, in degrees, Fahrenheit.
1............	80	107	75	27
2............	76	108	43.5	32
3............	76	102	27.5	26
4............	80	89	75	15
5............	76	96	18	20
6............	76	88	10	12
7............	78	90	23	12
8............	76	86	10	10
9............	78	84	23	6
Averages.........	77.4	94.5	33.6	17.1

center of the pier, Nos. 6 and 7 in the center of the arch halfway between the springing line and the crown, and Nos. 8 and 9 in the center of the arch at the crown. The arch is not exposed on its upper side, as there is an earth fill above it. Coils Nos. 6 and 7 are about 9½ in. from the surface, and Nos. 8 and 9 about 8 in., consequently the heat generated by the setting would be dissipated much more rapidly at Nos. 6, 7, 8, and 9, than at the others. Conversely, these four reach temperatures much lower in winter and somewhat higher in summer than the others. As the upper surface of the arch is protected by an earth fill, the range there is probably not as great as it would be if the arch had open spandrels, therefore the variations are on the conservative side.

It is evident, in this case, that the concrete set and the arch took its shape at a temperature of more than 90 degrees. Any deviation from this temperature, then, would cause stress in the arch. As all the thermometers in the arch itself reached zero, or went below zero, during the following winter, the arch was subjected to a stress due to a variation of more than 90° from the normal. The arch was designed for a variation of 40° each way from the normal, therefore it was subjected to 2¼ times as great a range as that for which it was designed. What is likely to happen in such a case? Probably the arch will crack, either at or near the abutments, or at the crown, or at both. As the arch at the abutments or piers is much deeper, and therefore much more rigid, than at the crown, the cracks are more likely to appear there. When the concrete cracks the stress is readjusted, and the arch has formed somewhat of a hinge for itself. As stated previously, the arch is not necessarily endangered by these cracks, as they cause a readjustment of the stress, but they are unsightly, and it is difficult to convince a layman that the arch is perfectly safe.

In his report Mr. Maxwell states that at that time there were no

Mr. Smith. cracks in the Walnut Street Bridge, but in the Squaw Creek Bridge there are unsightly cracks at the exact point of computed maximum stress at three of the four corners of the bridge. He also states that in the Locust Street Bridge, Des Moines, Iowa, there are cracks at the crown of each of the shore spans.

As all these arches were designed for a variation of 40° each way from the normal, Mr. Maxwell concludes that for northern latitudes a variation of this much should be allowed, and that:

"Particular circumstances may demand that a greater variation be used for drop in temperature to prevent the appearance of cracks."

This conclusion was reached by the writer some years ago. If, in the face of these facts, an engineer is willing to design a bridge for a variation of only 20° each way from normal, as Mr. Quimby seems to propose, and then lets the bridge crack and form its own hinges, the writer freely admits that it will be a cheaper bridge than a three-hinged arch designed for reasonable stresses.

It is admitted that, if a stone hinge be used in an arch, the friction of the hinges should be taken into consideration in computing the stress due to change of temperature, but with a metal hinge, such as that in the Traver Hollow Bridge, the amount would be so small as to be negligible. The writer would like to call Mr. Janni's attention to the details of these hinges and the arch rib, Figs. 10, 11, and 12. He will see that the span of the arch is 200 ft., and that the diameter of the pin at the abutment hinge is about 3 in., and at the crown 2 in.; therefore the maximum deviation of his lines, P_1 and P_4, would be 10 in. in 110 ft., which is certainly not very serious, to say the least. The pressure on these pins is about 12 000 lb. per sq. in. The socket for holding the pin is very heavy and rigid so as to cause very nearly an even distribution of pressure on the pin and a lining is cast in the socket and then finished to fit the pin accurately. The metal of the body of the hinge is cast steel having an elastic limit of 40 000 and a tensile strength of 76 000 lb. per sq. in., an elongation of 20% in 2 in., and a reduction of area of 41.5 per cent. A sample of this steel, ½ in. thick, was bent 180° around a diameter of 1½ in. without a sign of cracking either on the outside or inside of the bent portion. The socket of the casting was lined with "Monel" metal by casting it in place and then machining it to fit the pin accurately. The pin was of the same metal, finished from a rolled bar. This metal is a natural alloy of approximately the following composition: nickel 68%, copper 31%, and iron 1 per cent. The writer has a sample of this metal which tested as follows: tensile strength, 100 000 lb. and elastic limit 71 600 lb. per sq. in., elongation, 35% in 2 in.; reduction of area, 54.7 per cent. Now, with this metal as a bearing for the hinges, even if there was the concentration of pressure which Mr. Janni thinks is so serious,

there would still be a factor of safety of 3 on the elastic limit of the material, and this, the writer thinks, is sufficient. Mr. Smith.

For the bending moment due to friction on the pin, the coefficient of friction of the pin and lining may be assumed at 0.20. The best authorities give this coefficient, for metal on metal, dry, as from 0.15 to 0.25; and, as this bearing is bronze on bronze, it will probably be nearer the lower limit, so that 0.20 should certainly be conservative. The total pressure on the pin is about 2 000 000 lb., therefore the bending moment of the friction at the surface of the pin, with the lever arm of $1\frac{1}{2}$ in., will be $2\,000\,000 \times 0.20 \times 1.5 = 600\,000$ in-lb., which would be the moment caused by the pressure line deviating from the center line by less than 0.3 in., as the pressure decreases as the distance from the pin increases.

Mr. Janni speaks of taking care of the stress due to dead load and that due to shrinkage by neutralizing their effects, as much as possible, by the method mentioned previously, that is, by building the concrete of the arch in sections and leaving key spaces between them to be filled in after the cement has set—in not less than 10 days, he states. His reference to neutralizing the dead load stresses is supposed to mean the stresses caused by the pressure line for dead load leaving the center line of the arch and thus causing a bending moment, as direct compressive stresses caused by the dead load, of course, cannot be neutralized in any way. All that is necessary to accomplish this is to design the arch so that the center line is the pressure line for the dead loads on the arch. When the dead load has been obtained, it is very easy to do this. As stated previously, this is probably as good a method as can be found, for a fixed arch, to neutralize the stresses due to setting, but is, certainly, very expensive; and it does not affect the stresses due to a change from the atmospheric temperature at which the arch was cast to the extreme variation either way.

In any of the Northern States the average temperature for June, July, and August is higher than 70°, and the winter temperature goes well below zero at frequent intervals. Therefore, in any open spandrel arch built in the Northern States during these three months, the drop in temperature will probably be—as a conservative estimate—at least 60 or 65 degrees. Mr. Maxwell has shown that, in Iowa, at least, the range will be much more than that.

If all arches could be built between March 15th and May 15th, or between September 15th and November 15th, this trouble would not occur, and a method of neutralizing the stresses due to setting, similar to Mr. Janni's and allowing a range of 40° each way from the normal, would probably insure the arch against cracks.

In Mr. Aylett's Fig. 8, an arch of I-section, there would probably be some danger of the flanges breaking off, as he suggests. These flanges should be at least twice as thick as shown by Fig. 8, as it is

Mr. Smith. not practicable to approach the thinness of the steel beam flange with concrete. The proportions used in the Traver Hollow Bridge are shown on Fig. 10.

The writer, however, cannot see that there is much danger of these flanges being broken off, as the load of the arch is carried to the falsework as a distributed load across its lower surface. The paneling of the sides is of light construction and is not independent of the falsework, therefore it has to move with it, and, as the movement of the falsework is caused by a corresponding movement of the arch, the whole structure must, of necessity, move together.

It may be of interest to state that the writer has a letter from the contractor who built the Traver Hollow Bridge, stating that no difficulty was experienced in its construction.

The writer will admit that, if piers are to be built in a flowing stream, in which there are quantities of tree trunks with the boughs all nicely trimmed off a short distance from the trunk, so that they can catch on anything with which they come in contact, as shown in Mr. Aylett's Fig. 9, there might be danger of a quantity of drift collecting around the piers. However, he does not believe that there is much danger of an object, after being deflected by the cut-water, being drawn in again quickly enough to be caught by the down-stream leg of the pier. He can see no danger whatever of a tree trunk being deflected by the cut-water and then running in again with its butt between the legs of the pier, and then swinging around and hanging there. The cut-water throws the water away from the pier, and, on account of this obstruction, the water will be slightly higher as it passes, and hence the velocity will be increased. Any object deflected by the cut-water will be carried away from the pier slightly by this elevation of the water, instead of being drawn in.

In streams subject to heavy floods, where any obstruction of the channel would be serious, it is probable that wide piers would be out of the question, and undesirable, no matter what the economy would be in theory. The writer does not wish to be understood as claiming that the braced pier shown should be adopted in all cases, by any means, for he believes that the conditions at every important bridge should be investigated carefully. He thinks, however, that the braced pier is worthy of investigation when the design for a bridge is taken up.

Mr. Mensch is mistaken if he thinks that the writer claims, as a general thing, that the concrete in a three-hinged arch is about 50% less than in one that is fixed, and no statement to that effect was made in the paper. This happened to be a case where the comparison was unusually favorable to the three-hinged arch, as the proportion of rise to span is almost 1 to 6. The writer used this arch because he had designed it some years ago for the Rye Outlet Bridge,

and had all the dimensions on hand, and wished to compare a three-hinged arch with it. There are five arches in the Rye Outlet Bridge, and this is the center span taken for a single-arch bridge. Mr. Smith.

It is an interesting fact that though this comparison is so favorable for the three-hinged arch for a single span, when it is applied to the Rye Outlet Bridge there is very little difference in cost, not more than about 4%, if one does not take into account the steel reinforcement. The reason for this is that the bridge piers are so high and the superstructure so massive that the total cost of the arch concrete is only 13% of the cost of the bridge, and $13 \times 0.48 = 6.25\%$ without allowing for the cost of the hinges. Taking these into consideration, the difference is reduced to about 4 per cent.

In stating the conclusions in the paper the writer did not mean to claim that every individual design would necessarily agree with these conclusions, but that they were true in the average case. He thinks that the paper has brought out a valuable discussion on a subject which, heretofore, has not been treated at length, and is glad that the matter was brought up. He has not found anything in the discussion, however, to cause him to change his conclusions, but has been further strengthened in them.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

Discussion.*

By William Cain, M. Am. Soc. C. E.†

William Cain, M. Am. Soc. C. E. (by letter).—Mr. Janni makes a perplexing statement when he says: "cos. $(\alpha + \beta)$ is purely a symbol of a certain operation to be performed, and is not an actual value; its value is given by cos. α cos. β — sin. α sin. β". Perhaps this is a *"lapsus calami"*. All our textbooks state that the cosine of any angle, as $(\alpha + \beta)$, is an actual value and give tables of the actual values for various values of $(\alpha + \beta)$. Also, such values are often represented by certain lines on a unit circle, by aid of which the formula quoted is derived. By a consideration of such line values, it is at once seen that cos. $(\alpha + \beta)$ approaches indefinitely cos. β as α tends indefinitely toward zero, and the same result follows from using the development of cos. $(\alpha + \beta)$. Mr. Cain.

In fact, take the equation,

$$P\,Q = \frac{v\,\alpha}{\text{cos. }\beta - \alpha \text{ sin. }\beta},$$

which Mr. Janni gives. As he knows, α sin. β is an infinitesimal, and can be neglected in comparison with the finite quantity, cos. β, when connected with it with either a plus or minus sign. Thus, as α tends toward zero, without ever becoming zero, the value of $P\,Q$ approaches $\dfrac{v\,\alpha}{\text{cos. }\beta} = v\,\alpha$ sec. β, which is exactly the formula derived by the writer.

The writer cannot agree with Mr. Janni that the direction of the bending stress on the section, $N\,I$, varies in the manner shown in

* Continued from February, 1914, *Proceedings*.

† Author's closure.

Mr. Cain. Fig. 9. It will presently be proved that the stress on the cross-section of an actual beam, varies continuously in amount and direction, so that no sudden change in direction of 90° at any point can occur.

This is true even at points of contact of wheel loads, for such loads are in reality distributed. So far as dams are concerned, the changes in the amount and direction of the principal stresses along horizontal sections have been determined, both theoretically and experimentally.

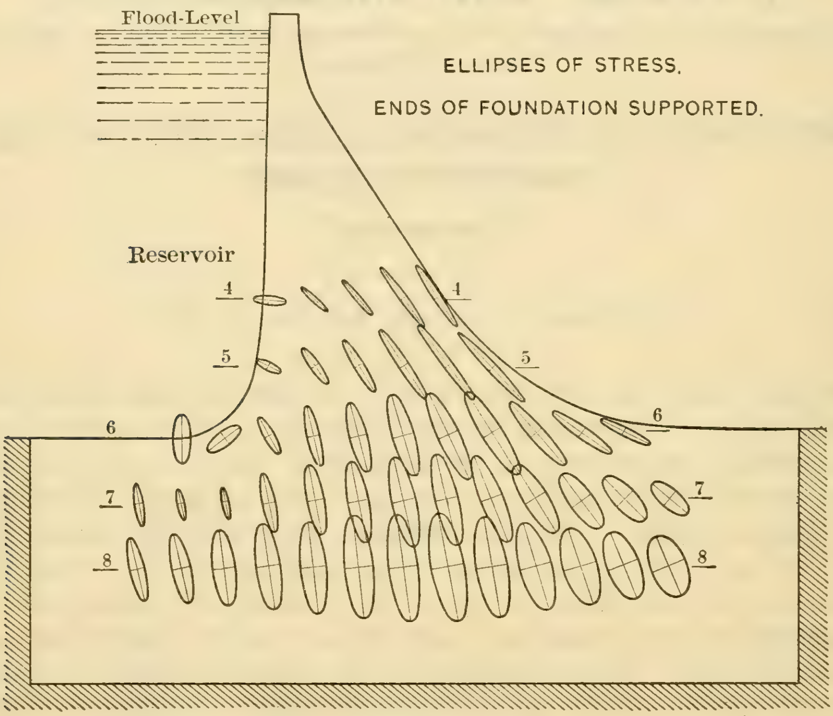

FIG. 13.

Reference has been made to the experiments on india rubber model dams by Messrs. Wilson and Gore. Fig. 13 is taken from their paper,* giving an account of these very interesting experiments. The unit stresses, in amount and direction, at numerous points in the dam, were determined from the distortions, and are represented by the ellipses of stress shown in the figure. The principal stresses, or those acting normally to their planes, are represented, in amount and direction, by the semi-axes of the ellipses. Of these, the maximum normal stresses are given by the semi-major axes and are those that specially interest us.

It will be observed that along any horizontal section, these unit principal stresses change gradually, in amount and direction. They are also observed to act parallel to the inclined down-stream face, very near that face. At the up-stream face, where the dam is subjected to

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, Session 1907–08, Part II.

water pressure, the direction of the principal stress is nearly normal to that face, to counteract the large normal pressure of the water. Mr. Cain.

The same general results were first found theoretically by Mr. Ernest Prescot Hill,* for a dam with the up-stream face vertical, and later by the writer† for the up-stream face battered; both writers adopting the usual approximate trapezoid law for the distribution of the vertical components of stress on a horizontal section.

Fig. 14 represents the changes in direction of the greater principal stresses, as computed by the writer on a horizontal section of a dam, 200 ft. below the water level, the reservoir being full. For reservoir empty, the principal stress at the up-stream face would act parallel to that face.

Now, suppose that, for a dam of the same size and shape, but of less specific gravity, and capable of exerting tension, the resultant on the horizontal section passes much nearer the outer toe or beyond it; then a part of the section would be in tension, but the theory quoted shows that the continuity of stress, in amount and direction, would be preserved, and that there would be no abrupt changes in either, in going from the up-stream to the down-stream face. This continuity is preserved, as the center of pressure on the base moves indefinitely beyond the outer toe, and hence it is true at the limit, when the weight of the dam is neglected and the horizontal section is subjected to a pure bending stress and shear, due to the water pressure alone. Exactly the same reasoning applies to the toe, Fig. 1, supposed to be homogeneous, if we combine with the upward acting soil pressure, the friction between the toe and the earth on which it rests. The resultant of this frictional resistance, acting to the right, and the vertical soil resistance, meets the section, *I N*, or *I N* produced; giving, as in the case of the dam, continuity of stress throughout the section. Now, let the friction force tend toward zero, then the point where the resultant meets *I N* produced recedes indefinitely from *I*, but the continuity of stress, in amount and direction, is always preserved, and hence it is preserved at the limit, where the friction force is zero and there is only bending stress and vertical shear on the section, *I N*.

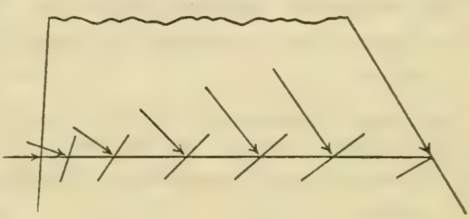

Fig. 14.

Thus, it has been rigorously proved that the bending stress in the toe, Fig. 1, whether or not soil friction is included, is inclined at the angle, β, to the horizontal at *N*, and this inclination is gradually and continuously lessened as we proceed down the section, *N I*, to *I*, where

* In a paper on "Stresses in Masonry Dams," *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 134.

† *Transactions*, Am. Soc. C. E., Vol. LXIV, p. 208.

Mr. Cain. it is zero. The same result, as to the continuity of stress, holds for any kind of a beam, subjected to any kind of distributed forces. This continuity of stress, in amount and direction, or the gradual change in both in passing along a joint, as proved above, seems to the writer to be axiomatic without any proof; but others may not think so, hence the proof.

It is difficult to see on what basis Mr. Janni assumes the discontinuity of 90° in the direction of the stress in Fig. 9. Presumably, the stresses act in the direction of the dotted lines. An obvious remark is, that if such directions were the true ones, then the arms of the corresponding stresses are less than those corresponding to the writer's hypothesis; so that even the writer's formulas would be on the danger side for the distribution of stress shown in Fig. 9; and the ordinary formulas, which Mr. Janni advocates, would be still farther on the danger side. There is thus an inconsistency which is fatal to Mr. Janni's contention.

The writer is satisfied that Mr. Janni has not read carefully the derivations of his equations or he would not have stated with reference to Equation 3:

"It is not clear to the writer, however, why the author gives a rather long equation (which, by the way, is not correct), when there are already several plain methods of finding that axis in a reinforced concrete beam."

The reference is to the neutral axis. It is hardly necessary to remark that if the "plain methods" are those pertaining to a prismatic beam (the only case ordinarily treated), they are entirely inapplicable to a wedge-shaped beam.

From overlooking this fact, Mr. Janni's Equation 23 is in error when β is not zero. The correct formula, assuming parallel compressive stresses, is given by Equation 15.

$$f_c = \frac{2\,M}{(b\,d)^2\,(k\,j)\,\cos.^2\beta}.$$

By the aid of Table 1, the values of f_c can be computed for various values of p and β, when b, d, and M are given.

In all the formulas, the value of β was supposed not to exceed about 45°, for reasons given, so that, when Mr. Janni, in Fig. 10, makes $\beta = 90°$, he is simply erecting a "man of straw" of his own in order to knock him down.

Further, in Fig. 10, to show a wedge-shaped beam, any dotted line must be extended to the left to represent the new upper surface, making the angle, β, with the horizontal. We are thus led to Fig. 1, where the resistance to bending along a vertical section is decreased, as compared with the case where $\beta = 0$, because the compressive stresses are now inclined to the horizontal and thus have shorter arms than for the case of the prismatic beam.

Mr. Cain.

The writer is pleased to know that Mr. Mensch thinks that the "assumption that the compressive forces are parallel to the compression face is certainly permissible in all cases where the percentage of reinforcement is low, say, less than 1 per cent." The next remark, that it "errs only slightly on the side of danger", presumably refers to the hypothesis of conservation of plane sections being adopted, as in the usual theory. Some experiments of A. M. Talbot, M. Am. Soc. C. E., seem to substantiate the hypothesis, those of Professor Schüle going the other way. For working loads, the hypothesis is universally used, as it is found to give safe results for reinforced prismatic beams, hence its adoption by the writer for the wedge-shaped beam, with the distinct proviso, however, that it was not to be used for large values of β.

Not only on this account, but principally because it was thought that the hypothesis of the compressive stresses, all acting parallel to the face in compression, departed too much from the truth for large angles, β was limited to about 45°, though, possibly, the limit might be extended to $\beta = 60°$, if the results are tempered with good judgment, which means a slightly increased factor of safety.

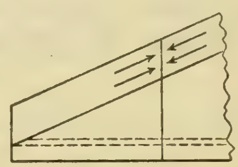

FIG. 15.

Mr. Mensch refers to the fact that in trusses with inclined compression members, the right section is used. It may be added that the stresses are taken parallel to the faces in metal beams with inclined flanges. Suppose a reinforced concrete T-beam, Fig. 15, with an inclined flange; would not a designer naturally take the compressive stresses in the flange as acting parallel to its surface? It seems absurd to take them as horizontal, particularly for the part of the flange on each side of the stem.

It is but one step now to the beam, Fig. 1, without a flange.

Mr. Mensch alludes to the fact that errors are sometimes made by using the factor, cos. β, in place of cos.^{2} β, in finding the compressive stresses. To emphasize this point, it should be remembered that A represents the area of a steel bar, taken at right angles to its axis, so that if f_s is the unit stress, the total stress in the bar is $f_s\ A$; but, considering a cylindrical "fiber" of the concrete, $N\ N'$, Fig. 2, the area of its section made by the plane, $I\ N$, is called $\Delta\, a$, so that the area of a section at right angles to the axis of the fiber is ($\Delta\, a$ cos. β), and the stress on the fiber is thus, $f_c\ .\ \Delta\ \alpha$ cos. β, f_c being the unit stress on the normal section.

For the fiber, $P\ P'$, Fig. 2, the stress is, similarly, $\frac{f_c}{kd_1}\ v\ .\ \Delta\ a$ cos. β, where $O\ P = v$, and the sum of such stresses over the area, $O\ N$, is.

$$C = \frac{f_c}{kd_1} \cos.\ \beta \sum_0^{kd_1} (v\ \Delta\ a) = \frac{f_c}{kd_1} \cos.\ \beta\ \frac{1}{2}\ b\ (kd_1)^2,$$

Mr. Cain.

since $\sum_0^{kd_1} (v \, \Delta \, a)$ is the statical moment of the area under compression about the neutral axis.

Therefore $$C = \frac{1}{2} f_c \, b \, . \, kd_1 \cos. \beta,$$

and the moment of C about G is,

$$M_c = \frac{1}{2} f_c \, b \, . \, k d_1 \cos.^2 \beta \, . \, D \, G \ldots\ldots\ldots\ldots\ldots (8)$$

For the special case given by Fig. 7, G coincides with I, and $D \, G = jd$, on putting $d_1 = d$. Therefore,

$$M_c = \frac{1}{2} f_c \, (kj) \, bd^2 \cos.^2 \beta = \frac{1}{2} f_c \, (kj) b \, p'^2,$$

where $p' = d \cos. \beta =$ the perpendicular from I on $N \, N'$ produced.

Mr. Mensch states that he has advised calculating a wedge-shaped beam, Fig. 11, for a section, $R \, R$. It is not clear exactly what Mr. Mensch means by the statement, for there are mathematical difficulties in the way where the section, $R \, R$, is not parallel to the line of action of the loads. However, accepting the hypothesis of parallel compressive stresses, the last equation gives the precise result,

$$M_c = \frac{1}{2} f_c \, (k \, j) \, b \, . \, \overline{R \, R}^2$$

as $p' = R \, R$, of Fig. 11.

It is to be observed, however, that k and $j = 1 - \frac{1}{3} k$, must be determined for a section (say a vertical section in Fig. 11) through the lower R, parallel to the loads and not for the section, $R \, R$, as follows from the derivation of the formula.

Formula 16 is in the simplest possible form, and by aid of Fig. 6, the values of k can be read nearly to thousandths, and the computation easily effected. As a matter of fact, the steel percentage for the toes, heels, or counterforts of retaining walls, is nearly always very low, so that the steel is the determining factor. In any case, both M_s and M_c should be computed from Equations 14 and 16, for assumed values of f_s and f_c, and the least of the two equated to M, the moment of the external forces; or, if possible, the steel section should be revised, because under-reinforcement is in most cases preferable to over-reinforcement, as it leads to a more progressive, and not a sudden, failure, in case of excess loading.

Mr. Nishkian raises an interesting point in connection with Fig. 4 (referring to the case of the heel-slab with the fillet), that the vertical component of the stress in the inclined rods, for the width, b, should be subtracted from the total external shear to give the shear in the concrete in section, $I \, N$. It may be observed, in reinforced concrete beams, that the bending of the beam entails the interaction of

steel and concrete, without which the steel would be under no stress whatever. Thus the tensile stress in any bar, at a section, is necessarily accompanied with a reaction or an equal bond shearing stress in the concrete, along the entire surface of contact of the bar, to one side of the section. The steel elongates under stress, and this elongation is resisted by the concrete and the total reaction, for the length of bar to one side of the section—the total bond stress—is exactly equal to the tension in the bar. Thus the stress in the bar does not relieve the concrete of shear directly. Even where there are hair cracks on the tensile side of the beam, the full bond stress is exerted between the cracks, giving rise to ordinary shears, and these in turn to indirect tensions and compressions in the parts intact, which play an indispensable rôle in transmitting stresses and maintaining the integrity of the beam.

Mr. Cain.

To deduce a formula for shear, only a part of the beam, such as is represented in Fig. 8, is considered. In an actual beam, as repeatedly pointed out, the compressive stresses are not all parallel to the face in compression, but their inclinations to the normal to $I\ N$ become less as the neutral axis is approached. In Fig. 8, the hypothesis of parallel compressive stresses is only used to establish approximately the point D, the point where the resultant, C, of the compressive stresses cuts $I\ N$. If C is now thought of as the resultant of the actual compressive stresses, having varying inclinations to $I\ N$, and calling β' its inclination to the normal to $I\ N$, the formulas which lead to Equation 17 are unchanged, provided we substitute β' for β. The angle does not appear in the final result. To realize further the conditions in an actual beam, tensile stresses in the concrete must be supposed between the neutral axis and the point where the hair cracks are experienced. Although, in the ordinary theory, such tensile stresses are ignored, yet it is seen, because they actually exist, that the section for maximum shear should strictly be taken at O, or at the neutral axis. Such ignored tensile stresses, though small, are nevertheless effective in changing the directions of the stresses in the concrete between the neutral axis and the point where cracks appear.

Formula 17 can be deduced in a different way from that given, which will introduce that interaction between the steel and concrete first alluded to. Let it be assumed that only shear of intensity, v, is exerted on a plane just below O, Fig. 8, that is, perpendicular to $I\ N$. Hence, adopting the previous notation pertaining to Fig. 8, the total shear on the plane is $v\ .\ b\ .\ dx$.

The bond stress exerted at the surface of contact of any bar, as $I\ I'$, throughout the length, $I\ I'$, is equal to the difference in the tensile stresses in the bar at I and I' and it acts in the direction, $I'\ I$. Hence the total bond stress exerted by all the bars is $(T' - T)$ in amount, and it acts in the opposite direction to T', if $T' > T$.

Mr. Cain.

If we call β_0 the angle made by the resultant, T (or T'), with the normal to $I\ N$, then $(T' - T)\cos.\beta_0 =$ the horizontal component of the total bond shearing stress exerted by all the bars, and this must equal the shearing stress, $v\ .\ b\ .\ dx$, exerted on the plane perpendicular to $I\ N$, just below O. Therefore,

$$v\ .\ b\ .\ dx = (T' - T)\cos.\beta_0.$$

Next, considering the free body, $N\ I\ I'\ N'$, held in equilibrium by the forces, V, V', C, C', T, T', acting as shown in Fig. 8, and taking moments about D,

$$(T' - T)\cos.\beta_0\ .\ \overline{D\ G} = V'\ .\ dx.$$

Eliminating $(T' - T)\cos.\beta_0$ between these two equations, dividing by dx, taking the limit as dx tends toward zero and V' tends toward V, and solving for v, we derive,

$$v = \frac{V}{b\ .\ \overline{D\ G}},$$

or Equation 17, which was first derived by a slightly different method.

At first sight, it might appear that the plane just below O, on which there is only shear, should be parallel to $T\ T'$, so that the shear on it should exactly equal the bond stress $(T' - T)$, which must be transmitted by ordinary shear to the neutral axis; but when we consider that along $O\ S$ (by the hypothesis), there is no bending stress, and hence no component of bending stress normal to $O\ S$, it follows that there can be only shear on $O\ S$, and hence a shear only of equal intensity on a plane normal to $I\ N$, say just below O, or between the neutral axis and the steel, if we ignore all tensile stresses in the concrete. The first equation above is thus justified. In the final equation for v, the whole external shear, V, appears, and there seems to be no justification for diminishing it by the component parallel to $I\ N$ of the pull in the rods.

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PAPERS AND DISCUSSIONS

A STUDY OF ECONOMIC CONDUIT LOCATION.

Discussion.*

By C. E. Hickok, Assoc. M. Am. Soc. C. E.†

C. E. Hickok, Assoc. M. Am. Soc. C. E. (by letter).—The writer wishes to confirm Mr. Hawgood's statement that the study of this particular conduit comparison was made under the latter's instruction. However, the diagram, Fig. 2, was originated and evolved by the writer. Mr. Hickok.

The writer did not credit tunnels with any saving in seepage, as he was unable to ascertain even an approximate figure for this saving. He agrees with Mr. Hawgood that there would be a saving, but this depends so much on the nature of the tunnel material, which may vary from a pervious soil to hard rock, that there was a hesitancy as to what value to place thereon. There might even be an addition to the discharge of the conduit in the tunnel due to underground waters.

Mr. Hawgood's diagram, Fig. 3, is interesting and is certainly more compact than that of the writer.

In regard to Mr. Stein's remarks as to the thickness of the concrete in the different types of conduit, it should be stated that these drawings, Fig. 1, were not inserted in the paper to illustrate construction details, but simply for pictorial purposes. The drawings were not necessary, and possibly should have been omitted. The writer hopes that no engineer would attempt to construct, for instance, a concrete flume similar to that shown on Fig. 1, without reinforcement. In regard to the 3-in. thickness of concrete lining in the tunnel, the section was supposed to be in solid rock, and no concrete lining was intended to be shown above the springing line. This escaped the writer's notice, due to the fact that he was more interested in the real subject than in construction details.

* Continued from February, 1914, *Proceedings*.

† Author's closure.

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PAPERS AND DISCUSSIONS

MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

By Messrs. Clarence S. Jarvis, and Gardner S. Williams.

Clarence S. Jarvis, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Lyman's paper, and the discussions regarding stream measurement, are of unusual interest, and will be of much practical value, especially in the irrigated districts. Mr. Jarvis.

Mr. Lyman has admirably disposed of the stupendous task of correlating a mass of data collected under a wide range of conditions, with varying viscosity of the water due to temperature changes and to chemical elements held in solution or suspension, the slight variations in the intensity of the force of gravity, and the personal equations of the observers, as disturbing factors.

The graphical comparison, portrayed so clearly on Plate LXXXI, of actual discharges measured by eminent experimenters with the quantities derived by the use of the diagrams for sharp-crested rectangular weirs without end contractions, proves to be a very satisfactory test of the curves on Plate LXXX.

If a single type of weir were to be adopted, either the Cippoletti or the sharp-crested rectangular weir without end contractions would doutbless serve best in irrigation practice.

One advantage of the Cippoletti weir is illustrated by Robert S. Stockton, M. Am. Soc. C. E., in discussing the "Management of Irrigation Systems".† A portable sheet-steel Cippoletti weir, of 24-in. crest, is used by the ditch rider to measure the water supply in the various small laterals.

* Continued from February, 1914, *Proceedings*.

† *Engineering and Contracting*, January 28th, 1914.

Mr. Jarvis.

In many irrigation districts where the slope of the country is slight, the writer has observed the need for a combination of measuring flume for current-meter gaugings and the approach to a suppressed weir. During the period of high water the canal or lateral is taxed to its capacity, and the obstruction required to give a free fall for such a stream over a weir is prohibitive. During this period current-meter gaugings are in order; and the rating curves for successive years need not change materially.

With the beginning of the low-water period, which, in the Great Basin region, is generally early in July, the duty of water is often increased two- or three-fold, and, with the reduced stream in the canal, it is possible to insert the weir crest in the measuring flume, thus providing a free overfall. The sediment does not interfere seriously after the high water has ceased; and the increased accuracy and reduced labor involved in the weir measurements recommend this method. Under these conditions the rectangular weir without end contractions provides the maximum width of crest and the minimum obstruction to the flow in the canal, and is much to be preferred.

In the writer's opinion, the data contained in Mr. Lyman's paper, and in the discussions, fill a long felt demand, and should wield a strong influence in the direction of higher duty for irrigation water, which will result from its careful distribution and judicious application on the irrigable land.

Mr. Williams.

GARDNER S. WILLIAMS, M. AM. SOC. C. E. (by letter).—This paper brings to the front once more the measurement of water over weirs, and embodies the results arrived at by the author in a study of most of the data available in 1905, supplemented by some investigations of his own of more recent date. It is to be regretted that the hitherto unpublished data of experiments involved in this paper were not presented in such a concise form as to have warranted publication. In physical investigation, the one thing above all others that is valuable is a correct record of the experiments and their results. Opinions, theories, and conclusions are likely to change as time goes on, but physical facts remain physical facts, and may be as valuable 100 years hence as they are now, though the experimenter's conclusions may be long discarded.

A considerable part of the experimental work discussed in the paper was performed in the Hydraulic Laboratory of Cornell University under the writer's direction, and much of it under his personal supervision and with his participation, and the data have received careful study at his hands independently of the author's work.

Our basic knowledge of the quantity of water flowing over weirs rests on the experiments of Francis, Fteley and Stearns, and Bazin, all of whom experimented on weirs of different dimensions, and read the head of water at different places or by different devices. No volumetric

measurements were made at Cornell prior to 1905, except in a few low head experiments wherein the inaccuracy of the volumetric measurement, on account of the large area and small rise in the measuring canal, was greater than that of any of the weir formulas could possibly have been. Since that date volumetric measurements have been made at Cornell over a weir 6 ft. long, and at the University of Utah, as described by the author. It is to be hoped that the data of the Cornell experiments may be presented in the discussion of this paper, as the volumetric determinations were probably more accurately made than in any other series of investigations by reason of the measuring basin being a vertical tube or stand-pipe 6 ft. in diameter and about 60 ft. high. Mr. Williams.

The builders of the Hydraulic Laboratory at Cornell made no provision for the reading of heads on the weirs contemplated. To cut chambers in the solid concrete of the walls and the rock adjoining it involved considerable expense, and when the first weir investigation was called for, that of the U. S. Board of Engineers on Deep Waterways, after some discussion, a device for communicating the head to glass tube gauges was adopted, consisting of a 1-in. galvanized-iron pipe, perforated with ¼-in. holes every 6 in. throughout its length, and placed transversely across the channel of approach, with the openings on the under side, and the pipe 8 in. above the bottom. Three such pipes were set for each weir, varying in distances from the base of the respective weirs. For this device there was then the unquestioned precedent of the similar apparatus at the Holyoke testing flume, which had been used there for many years, and is still. Before this series of experiments was completed, however, the possibility of an erroneous indication of head suggested itself, and, to test the matter, similar pipes were set in the floor of the timber flume leading to the lower weir, with the openings on the top and just flush with the bottom of the channel. The result was somewhat startling, for it was found that, for heads of about 4.50 ft., the original device was giving an indication about 0.30 ft. lower than that shown by the openings in the bottom of the channel. A series of simultaneous observations on the two devices was made, from which the former were corrected to the latter, and it was assumed that this latter device corresponded in its indication to those used by Francis, Fteley and Stearns, and Bazin. At the upper weir it was not so easy to get a series of openings flush with the bottom, as that would have necessitated cutting into the concrete, and after some study of existing data, the so-called standard tube-piezometer, described by the author on page 1577,* was adopted, and on the strength of the experience of Desmond FitzGerald, Past-President, Am. Soc. C. E., with a perforated pipe in a water main,† it was as-

* *Proceedings*, Am. Soc. C. E., for September, 1913.
† *Transactions*, Am. Soc. C. E., Vol. XXXV.

Mr. Williams.

sumed that the indication of this device corresponded to that of an orifice at the bottom of the side-wall similar to the one used by Bazin. With the correctness of this assumption, the writer was never entirely satisfied, but, during his connection with the Laboratory, from the conclusion of the first series of investigations, this device was used at the upper weir in all experiments and very carefully observed, all other devices being compared with it. During the earlier work the upper weir was taken as the standard, and the discharge over it was computed by Bazin's formula, using the indication of the standard tube for the head. In the spring of 1901 the lower standard weir was installed, and the head on it was usually read by the 13-ft., three-tube-piezometer, described by the author on page 1585.* The indication of this apparatus was never considered quite as reliable as that of the standard tube because of a greater likelihood of the presence of air in the pipe connecting it to its gauge. The lower sharp-edged weir was designed to be as nearly as possible a duplicate of the 19-ft., Farm Pond weir of Fteley and Stearns, it being 6.65 ft. high, or $\frac{1}{10}$ ft. higher than the Farm Pond weir, and 16 ft. long. During an extended series of experiments the head on this weir was read at a point 6 ft. up stream from the crest and just below its level, this coinciding with the practice of both Francis and Fteley and Stearns.

To those interested in the more precise measurement of water, the data obtained from the resulting use of different devices for reading head make the most valuable part of this paper, and are embodied in Tables A and B.

The first investigations, as to the effect on the apparent discharges, by different methods of reading the head on weirs, were made by the late James B. Francis, Past-President, Am. Soc. C. E., and are presented in "The Lowell Hydraulic Experiments".† These experiments are discussed by Messrs. Fteley and Stearns.‡

The next investigation of the subject of reading heads, so far as the writer has discovered, was that of Hiram F. Mills, Hon. M. Am. Soc. C. E.,§ which appeared to demonstrate that an opening in the wall of a channel, in order to transmit a pressure corresponding to the height of the surface of flowing water opposite, must be flush with the side, and the connecting pipe must be at right angles thereto.

The subject of indications of head, particularly as applied to closed channels, received further discussion, but without experimental evidence, in the paper on "Experiments Relating to Hydraulics of Fire Streams" by John R. Freeman, M. Am. Soc. C. E.||

* *Proceedings*, Am. Soc. C. E., for September, 1913.

† Edition of 1883, page 137 *et seq.*

‡ *Transactions*, Am. Soc. C. E., Vol. XII.

§ *Proceedings*, American Academy of Arts and Sciences, Vol. VI, New Series. Boston, 1879.

|| *Transactions*, Am. Soc. C. E., Vol. XXI.

Mr. Williams.

In the paper on "Flow of Water in 48-In. Pipes" by Desmond FitzGerald, Past-President, Am. Soc. C. E.,* a statement appears covering the similarity of indication of a perforated pipe laid longitudinally on the bottom of a water main, and a series of circumferential orifices communicating to a chamber surrounding the pipe; and some further investigations along similar lines are presented in the paper on "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah".† Beyond this, the effect of different methods of reading head does not appear to have attracted special notice until the writer called attention to it and presented experimental data bearing thereon, in the discussion of the paper, "On the Flow of Water over Dams," by the late George W. Rafter, M. Am. Soc. C. E.,‡ since which time nothing further has been presented in the publications of this Society, at least as applied to weirs, until the paper under discussion.

In these days, particularly in water-power work, where the competition between various manufacturers leads to the proposal of bonuses and forfeits, based on as small variations of discharge as one-tenth of 1%, exact knowledge of the influence of every departure from the standard of reference is of inestimable importance. In a recent test of water-power apparatus, a bonus of several thousands of dollars was apparently won, a considerable part of which was due to the introduction of irregular methods of observing the head on a weir, without a consideration of their influences.

Theoretically, what is desired in reading the head on a weir is to locate the elevation of the surface of the water, but, with channels of even very moderate widths, this will not be the same at all points across the stream, so that the measurement of it by even several tapes in a cross-section may not give an exact indication of the average head, and if it would, in no experimental investigation wherein the discharge of large weirs has been determined volumetrically, except those at the University of Utah, has such a process been used. The methods of transmitting the head through orifices in the side-walls or in plane surfaces parallel to them, as adopted by the leading investigators, is open to the criticism that the pressure thus transmitted is influenced by the velocity past the orifice to an extent that becomes decidedly appreciable at high heads. If, in order to avoid high heads, long weirs are used, the question then arises as to the applicability of an indication transmitted through the side-wall, to the establishment of the average head across a stream, say, 100 ft. wide. Bazin apparently conceived that the head should be transmitted from a location where the velocity would be the lowest possible, and hence selected a point well up stream from the pressure angle and at the bottom of the

* *Transactions*, Am. Soc. C. E., Vol. XXXV.
† *Transactions*, Am. Soc. C. E., Vol. XLIV.
‡ *Transactions*, Am. Soc. C. E., Vol. XLIV.

Mr. Williams.

side-wall. For long weirs it seems to be necessary to supplement this orifice by openings in the bottom of the channel at frequent intervals across it, and perhaps the most satisfactory device is a chamber extending across the bottom of the channel and communicating with it through medium-sized orifices at frequent intervals. Although it may be assumed that the indication of head transmitted through such a device cannot be very different, for short weirs, from that used by Bazin, it is nevertheless quite certain that it will not exactly coincide.

Different forms of transverse and longitudinal pipes, variously perforated, are frequently used, but both devices, if outside the pressure angle, are sure to read lower than an orifice in the wall near Bazin's location, on account of the reduction of pressure due to increase of velocity, as the location departs from the wall and bottom. In the experiments presented by the author, the standard tube, although intended, and originally supposed, to measure a head corresponding to that indicated by Bazin's device, fails to do so, and actually gives a lower one. The 11-ft., wall-piezometer coincides more nearly with a Bazin indication, but is still, in all probability, too low, as by the author's equation the Bazin reading is more than 5% higher than the 15-ft. tape.

Being taken in the open channel, the tape readings are far from satisfactory for fine work. At low heads, with smooth water, the tendency of the observer is to cut the surface too deeply, thus giving a low head, and, for high heads and rough water, the observer invariably reads too high. The same applies to observations with the point-gauge in running water. This accounts in part for the irregularity of the lines representing tape readings. Although, if the water is stilled, a skilled observer can do excellent work with a tape or any other form of point-gauge, the ordinary student, dealing with the rough water encountered at high heads, comes very far from getting ideally consistent results.

In order that the record may be complete to date, the writer presents the following data of comparisons of different methods of reading heads.

I.—At Cornell University Hydraulic Laboratory, 1899. Upper Standard Weir, 16 ft. long.

A = Standard tube, by double-tube gauge, as described by the author.

B = Upper transverse pipe, by double-tube gauge.

This was a 1-in. pipe perforated on the bottom with ¼-in. holes, 6 in. apart, extending across the canal parallel to the weir, 8 in. above the bottom and 27 ft. up stream from the 13.13-ft., high, standard weir.

C = Middle transverse pipe, by double-tube gauge.

This was in all respects similar to B, except that its location was 10 ft. up stream from the weir.

A and B and B and C were observed simultaneously on the same scale. Heads are from mean curves of the observations.

Mr. Williams.

HEADS.

A	*B*	*C*
0.164 ft.	0.164 ft.	0.164 ft.
0.492 "	0.484 "	0.492 "
0.984 "	0.970 "	0.985 "
1.476 "	1.461 "	1.495 "
1.968 "	1.938 "	1.993 "
2.460 "	2.410 "	2.489 "
2.950 "	2.860 "	2.965 "

In these experiments it appears that *C* was probably within the pressure angle.

II.—At Cornell University Hydraulic Laboratory, 1899. Lower, Sharp-Edged Weir, 6.56 ft. long.

D = Flush pipe, by double-tube gauge.

This was a 1-in. pipe perforated on top with $\frac{1}{4}$-in. holes, 6 in. apart, and set with its top flush with the bottom of the flume, parallel to and 37 ft. up stream from the 5.2-ft., high, sharp-edged weir.

E = Upper transverse pipe, by double-tube gauge.

This was similar to *B* and 37 ft. up stream from the weir.

F = Middle transverse pipe, by double-tube gauge.

This was similar to *B* and 19 ft. up stream from the weir.

G = Middle flush pipe, by double-tube gauge.

This was similar to *D* and 19 ft. up stream from the weir.

D and *E*, *E* and *F*, and *D* and *G*, were observed simultaneously on the same scale. The heads for *D*, *E*, and *F* are from mean curves, those for *D* and *G* are from direct observations.

HEADS.

D	*E*	*F*	*D*	*G*
0.920 ft.	0.912 ft.	*F* = *E*	0.744 ft.	0.745 ft.
1.010 "	1.002 "	Nearly.	0.913 "	0.914 "
1.061 "	1.050 "		1.086 "	1.088 "
1.632 "	1.614 "		1.292 "	1.292 "
1.668 "	1.642 "		1.441 "	1.442 "
1.970 "	1.934 "		1.632 "	1.631 "
2.566 "	2.480 "		1.810 "	1.810 "
3.380 "	3.270 "		1.811 "	1.807 "
3.820 "	3.655 "		2.240 "	2.239 "
4.160 "	3.970 "		2.642 "	2.643 "
4.310 "	4.080 "		(3.470) "	(3.460) "
4.820 "	4.560 "		4.270 "	4.277 "
5.040 "	4.720 "		4.670 "	4.681 "
5.450 "	5.090 "			
5.560 "	5.180 "			

Mr. Williams.

III.—At Cornell University Hydraulic Laboratory, 1899. Lower Experimental Weir, 16 ft. long. U. S. Deep Waterways Section.* 1:1 up stream slope tangent to 3.3-ft. radius curve at top; height, 6 ft.

H = Longitudinal pipe by double-tube gauge.

This was a 1-in. pipe similar to *B*, but laid on the bottom parallel to its axis and about 20 ft. up stream from the weir.

I = Series of ¼-in. orifices, 6 in. apart, throughout the length of the crest, by double-tube gauge.

J = Tape, nailed on the wall vertically from the crest of the weir, and read by contact of water surface.

H, therefore, indicates the head on the weir;

I, the pressure in the sheet at the crest; and

J, the vertical depth of water at the crest.

HEADS.

H	*I*	*J*
0.848 ft.	0.525 ft.	0.582 ft.
1.481 "	0.848 "	1.035 "
2.170 "	1.099 "	1.540 "
2.753 "	1.208 "	2.010 "
3.180 "	1.242 "	2.304 "

IV.—At the Holyoke Testing Flume, 1900. Sharp-Edged Weir, 20 ft. long.

K = Francis orifice, by double-tube gauge.

This was a ¼-in. opening flush with side-wall of canal, 6 ft. up stream from, and 0.34 ft. below, the level of the crest of a weir 5.85 ft. high.

L = Holyoke pipe, by double-tube gauge.

This was a 1-in. brass pipe perforated on its under side with ⅛-in. holes every 2 in., and set 1 ft. above the bottom of the channel, parallel to and 10.2 ft. up stream from the 5.85-ft. high weir.

K and *L* were observed simultaneously on the same scale.

HEADS.

K	*L*
0.392 ft.	0.392 ft.
1.092 "	1.087 "
1.358 "	1.352 "
1.548 "	1.536 "
1.721 "	1.710 "
1.873 "	1.857 "
1.994 "	1.971 "
2.079 "	2.055 "

* *Transactions*, Am. Soc. C. E., Vol. XLIV, p. 284.

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V.—At the University of Michigan, 1912. Special weir, 1.865 ft. long, with one end contraction of 1.08 ft. on side next orifices and gauge-reading devices. Weir, 11.46 ft. high.

M = Francis orifice, by hook-gauge.

This was $\frac{1}{4}$-in. in diameter, flush with side-wall, 6 ft. up stream from, and 0.34 ft. below, the level of the crest, connected by $\frac{3}{4}$-in. pipe to a gauge chamber 15 in. square.

N = Bazin orifice, by hook-gauge.

This was 4-in. in diameter, 7.32 ft. up stream from the weir, with its center 0.4 ft. above the bottom of the channel, connected by 4-in. pipe to a gauge chamber 15 in. square.

P = Point-gauge, its contact with the water surface observed electrically by the flash of a lamp.

This was situated in a 10 by 12-in. rectangular box of 2-in. plank attached to the side-wall above the Bazin orifice; the bottom of the box was fully open and 0.79 ft. above the level of the crest, and its down-stream face was 7.47 ft. up stream from the weir. The point was located 7.82 ft. from the weir, and 0.25 ft. from the wall side of the box.

The reading of the several gauges, corresponding to the crest of the weir, was determined both by wye-leveling and by water levels from a hook-gauge attached to the weir crest.

For M and N the crest reading by the water level was 0.005 ft. higher than by the wye-level, and for P, 0.011 ft. higher. This indicates that the determination of crest readings by wye-leveling results in slightly too high a head being used for the discharges. Similar results were obtained at the Holyoke testing flumes.

The heads observed were:

M	N	P
1.1551 ft.	1.1623 ft.	1.1758 ft.
1.3575 "	1.3883 "	1.3918 "
....	1.4572 "	1.4602 "
1.4386 "	1.4435 "	1.4456 "

It is to be noted that, on account of the small size of the opening for M, the water rose very slowly in the gauge tank, and the low reading of the second, as well as the relatively high reading of the fourth, observation may be partly due to that cause. The comparisons of N and P are believed to be very accurate.

As to the comparisons in the author's Tables A and B, it may be said that at the lower weir those of the Francis, float, and bottom piezometers, when more than one was used, being read by the same observer on the same scale of a double-tube gauge, should have been,

Mr. Williams.

and in all probability were, very carefully observed, and errors in one would be duplicated in the other.

The same is true as to comparisons at the upper weir between the 6-ft. longitudinal and 6-ft. wall, the 6-ft. longitudinal, the 11-ft. wall, the 2-ft. wall, and the crest piezometers which were observed in pairs on double-tube gauges, and the connections shifted so that the 6-ft. wall and 6-ft. longitudinal were observed with each other, and one of these two with all the others.

Tape and point-gauge readings, the standard tube, Bazin hook, and the longitudinal piezometers at the lower weir were necessarily read by different observers, and their differences involve a personal equation as well as possible inaccuracies in establishing the zeros or crest readings of the scale. The greatest care was taken in observations on the standard tube and the Bazin hook, and the work was very well done on those instruments and may be safely accepted. As to the tapes and the other apparatus, the writer does not feel quite so confident.

Table 0 consists of a series of coefficients to apply to the discharge of broad-crested and irregular weirs of a height of 11.25 ft. to give the discharge of weirs of other heights. This table is based on the effect of changing heights of sharp-crested weirs. Although the data used by the author seem to be the best we have, it is, nevertheless, to be remembered that the effect of a change of height on a broad-crested weir may be less than on a sharp edge, and hence the changes indicated by Table 0 may be too great. In other words, it seems probable that for flat-crest weirs higher than 11.25 ft., the discharge may be greater than that obtained in Table 0, and for similar weirs of less height than 11.25 ft., the discharge may be less than that indicated by the table, both by a small amount.

The evidence of Plate LXXXI, that all sharp-edge weir experiments fall substantially within 3% of a mean curve, is certainly valuable.

The curves shown on Plates LXXXIV and LXXXV are to be considered as approximate near the points where the lines deflect. There are conditions under which abrupt changes of condition arise in the discharge of weirs, but certainly nothing of the sort should appear in the standard weir at the points of deflection of its lines on these plates. More numerous experiments in the region of the deflections would show a gradual, not a sudden, change in the direction of the line.

In discussing the indication of the Francis orifice at the Lower Cornell Weir, on page 1548,* the author calls attention to apparently erratic readings, and offers as an explanation the possibility of an oblique current. This orifice was ¼-in. in diameter and near the center

* *Proceedings*, Am. Soc. C. E., for September, 1913.

Mr. Williams.

of a smooth brass plate, 12 in. long and 6 in. wide, set flush with the timber forming the side of the canal. The likelihood of a current causing the discrepancy is remote, but a much more probable explanation is that the zero of the gauge was very close to the floor, and the observer had to lie in a very cramped position to observe low heads. All readings of the scale below a head of 1 ft. are likely to be low by a small amount, including the determination of the zero, on account of the observer not getting his eye on a level with the water in the glass. The effect of this at low heads would be to give a reading ranging between 0.004 and 0.002 ft. in error, which might be either high or low depending on the relative care with which the crest reading and low-head observations were made. From Plate XCIII it would appear that the low readings were high, which means that the zero was more accurate than the low-head readings.

The author's comparison of these indications by the approximate straight lines is open to some criticism, as the deviation of the line at some points from the true locus may be greater than that of the observations it is intended to harmonize.

On page 1579* attention is called to a note in the Laboratory Record, that the 6-ft. tape at the upper weir was below the beginning of the surface slope, and, therefore, was reading low. The statement is correct, and all the 6-ft. piezometers were similarly affected, and the fact stated, that the standard tube, 26 ft. up stream from the weir, was giving a lower reading than these, is to be explained by the relatively higher velocity past its orifices than past those near the wall. The reduction of pressure due to velocity past openings was imperfectly appreciated by the writer until experiments in a closed pipe showed that the pressure indicated by an orifice in a surface parallel to the axis decreased from the walls to the center when water was flowing.

In the writer's reduction of the experiments at Cornell, the results of which appear in the volume of Hydraulic Tables, of which he is a joint author,† the various discharge curves were compared with each other, with the sidelight of his personal knowledge of the peculiarities and characteristics of the several observers and apparatus involved. No attempt was made to establish the actual discharge in any case, but only to establish a ratio by which the discharge of a similarly dimensioned sharp-edged weir should be multiplied to give the discharge of the irregular weir. For himself, it involves less labor and conduces to accuracy to take the discharge of a sharp-edged weir from a reliable table and increase it by a known percentage than to compute the discharge *de novo* by using a special coefficient. If others prefer different methods, he has no objection, always bearing

* *Proceedings*, Am. Soc. C. E., for September, 1913.

† "Hydraulic Tables", Williams and Hazen, John Wiley & Sons, New York City.

Mr. Williams.

in mind, however, that the base discharge of the sharp-edged weir may be somewhat in doubt for cases beyond the range of experiment.

To the preparation of this paper the author has devoted a great amount of time and labor, a fact which no one more fully appreciates than the writer. He has assembled and presented in new form much valuable information, and for this is entitled to the thanks of all interested in the subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

By Messrs. V. H. Hewes and Lazarus White.

V. H. Hewes, M. Am. Soc. C. E.—The speaker would be glad if some member of the Board of Water Supply, who is familiar with the grouting work used in the siphons on the line of the Catskill Water Supply for New York City to check the excessive inflow of water during construction, would describe the method used in that work. Mr. Hewes.

The speaker was granted the privilege of visiting the inverted siphon under the Hudson River where a heavy inflow was encountered, which threatened to drown out the pumps, although extra pumping facilities had been provided to care for such an emergency. A bulkhead was placed in the drift near the heading; pipes were introduced through the bulkhead, and grout was pumped through them, thus effectually reducing the inflow. After the bulkhead was removed and the work of drifting was continued, it was found that the cement had filled the rock seams both laterally and longitudinally. A sample of porous rock was shown which had been taken from another siphon on the line, where the grouting method was used to check the inflow. This sample seemed to be thoroughly impregnated with cement.

A description of the method used to relieve the water pressure on the outside of the concrete lining until the concrete had thoroughly set at points where there were small seams in the rock walls, and the subsequent grouting of these seams, would be exceedingly interesting.

Lazarus White, Assoc. M. Am. Soc. C. E.—It is interesting to compare the results at the Estacada Dam with those obtained on the Catskill Aqueduct. The Ashokan and Kensico Dams have very deep concrete cut-offs. Some grouting was done below these cut-offs, but only Mr. White.

* Continued from March, 1914, *Proceedings*.

Mr. White. as a secondary matter. In the sinking of the shafts there was considerable grouting, which afforded a good opportunity to see what it would do, because, in sinking a shaft, the portion grouted is subsequently excavated. The grouting of tunnels during driving also gives very definite information. The grouting work on the Catskill Aqueduct has been going on for 5 or 6 years, and has been watched for this entire period, so that observing engineers have come to some conclusions as to its results. These conclusions are remarkably similar to those expressed in this paper, which is exceptionally instructive.

The lack of success of a portion of the Estacada grouting is frankly admitted. It seems that in the last few years grouting has been made a sort of fetish; that is, people have been expecting too much from it. At best it is an uncertain proposition, and should be secondary to the customary work. The same applies to the excavation of tunnels. More water is cut off by the concrete lining than by the grouting. Grouting at times is wonderfully successful, but the conditions then are rather favorable; where the rock is hard, with definite splits and channels, the grout can be forced into it readily, but where it resembles the volcanic rock described by Mr. Rands the grout cannot be forced in thoroughly, but takes its own channels. It may run a long way from the part to be grouted, and on excavating this part, very little of it will be found. No one knows where it went—all that is known is that one has paid for it. At some shafts on the Catskill Aqueduct holes have been repeatedly grouted, and when the excavation was extended through the grouted zone the grout could hardly be seen—the same being even more true of grouted tunnels.

As a general proposition, the grout goes into places which the engineer has good reason to know exist, that is, into definite cracks or openings. At some shafts grouting has been very successful, but as already mentioned, the conditions there were favorable. Where unfavorable conditions were found, for instance, resembling those at the Estacada, grouting was not very effective. Judging by experience on the Catskill Aqueduct, the conclusions of this paper are entirely sound.

Grout always seems to obtain some set, but if it is forced into a place where there is water under pressure, and if the water is circulating, it will carry the grout out of the seams before it has a chance to set. Grout has been chased over tunnel arches, up and down, for hundreds of feet. Before the grout can set, it may be forced out, and pipes and seams which were thoroughly grouted may become empty, but if the grout is kept in place long enough, it will set no matter how much water is put in. This seems to be the speaker's experience.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

Discussion.*

By Messrs. E. W. James, Arthur H. Blanchard, and Spencer J. Stewart.

E. W. James, Assoc. M. Am. Soc. C. E. (by letter).—The author has been particularly fortunate in his studies of sand-clay mixtures for road surfacing, in that his investigations have been made at the best point in all North America for such a purpose. The region especially concerned in the sand-clay type of construction is the Coastal Plain between the Upper Piedmont and Tidewater. According to State lines, therefore, Georgia, and especially North Georgia, is in the center of the sand-clay territory. The best natural mixtures are probably found wherever the Orangeburg sands of the Lafayette formation are prevalent or common. Having confined his investigations to these conditions, the author's conclusions are doubtless stated a little more strongly than they would have been had his experience or studies covered a wider territory. It is safe to state that sand-clay roads can be built in few sections of the United States in such a way as to give as great a degree of service as in Clarke or Elbert Counties, Georgia. Mr. James.

For several years the writer has had a wide experience in sand-clay work throughout all the Coastal Plain from Virginia to Southern Texas, and recognizes the importance of this type of construction. For a large section of the Southeastern States it is the only possible type of improved road for a large mileage of the country highways. The importation of stone is prohibitive because of excessive cost; shell is too friable and soft; brick, even on a natural sand foundation, is too expensive; and concrete is practically out of the question because of the cost of both sand and aggregate.

* This discussion (of the paper by John C. Koch, Assoc. M. Am. Soc. C. E., published in February, 1914, *Proceedings*, and presented at the meeting of March 4th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. James. For many counties in the Southern States, therefore, the sand-clay road is the only improved type within the means of the locality, and the only one economically warranted by the conditions of traffic at the present time and probably for many years to come. In many sections, especially in the Tidewater and Coast counties the natural soils are prevailingly sandy. It is difficult to find good clays. The sand beds are as bad or worse than the clay mud of the uplands, and, moreover, they are bad, not only in winter, but all the year round. For this reason the sand-clay road is especially valuable in these sections. The natural soil is well drained, and the latitude precludes any lasting or great degree of frost. The question in these regions is not commonly to select materials that will produce a sand-clay of certain stability, but to use the materials at hand so skilfully as to produce alleviation of the almost intolerable conditions presented by the sandy roads.

In the counties along the coast, and usually for about 100 miles inland, the sands are fine and often floury; all clay, except occasional small pockets of white or gray pipe-clay, or a mottled red and gray variety, carries a large percentage of this fine sand. Consequently, the field methods of testing the clays described by the author would, in the writer's experience, result in rejecting a majority of those locally available, and of discarding all but the rarest deposits of sand.

The author's conclusions, however, are recognized as well founded, especially in regard to the value of coarse sand.

For sand-clay mixtures, the writer has frequently used a most simple field test, which has served his purpose well. The sources of material are located and selection is made, depending usually on the length of haul and the depth of the deposits beneath the surface. The immediate problem is to determine the best mixture of the materials selected.

Typical samples are taken of both sand and clay. Mixtures are made, ranging from 1 part sand to 3 parts clay, up to 3 parts sand to 1 part clay, or sometimes beyond these limits, if the materials appear to warrant it. These mixtures should be made to vary by one-half of 1 part, 1 : 3, 1 : 2½, 1 : 2, etc., 1½ : 1, 2 : 1, 2½ : 1, etc., and should be worked up with water into putty-like masses.

From each test, mix a small sample of from 1 to 2 cu. in., cut out with a small measure. The writer has found a small medicine glass, or even a large brass thimble, handy. It is only essential to get equal samples from each test mix. These samples are then rolled between the palms of the hands into reasonably true spheres and placed in the sun to dry. Some designating marks may be scratched on them. When thoroughly baked, they are placed in a circle in a flat pan or dish, and enough water is poured in the pan to cover them, care being taken not to pour the water on the samples.

Slaking will begin at once. The lapse of time found by Mr. Koch with his compressed specimens is not found at this stage. The slaking, however, will proceed at different rates. The sandy specimens will break down first, those with excessive clay will disintegrate second in order, and those having about the proper proportions will act more slowly. Usually, there will be one or two that determine the proper proportions of the materials, and, in the writer's experience, these will usually lie together in the series of test mixtures. Mr. James.

A supplementary test of some value can also be made on the dry spheres. Lightly rubbed with the thumb, those having too much sand will break down rapidly. Those having too much clay will soon begin to "dust" away, while those having the most stable mixtures will assume a slightly glazed effect under the light rubbing, due to the moisture and oil of the skin. These two tests will not give the same results. The dry test will indicate a mixture richer in clay as the better one, and the wet test will indicate a sandier mixture. The sample indicated as satisfactory under the wet test that lies between the other two will prove best in service.

These extremely simple tests do not determine the sand or clay content of the mixture, for the clay selected almost always contains considerable sand, and this, in most cases, contains silt or clay. The tests serve, however, to fix the actual values in the mixture of the pit run, as represented by the samples.

Arthur H. Blanchard, M. Am. Soc. C. E.—The speaker's discussion will be limited to calling attention to certain features of the construction of sand-clay roads which it is believed the author should include in his very valuable contribution to the literature on this subject. Mr. Blanchard.

The author states that the first cost averages $500 and the annual cost of maintenance is $5 per mile; and then emphasizes the fact, agreed with by the speaker, that there is a place for the sand-clay road. Very little information, however, is given with reference to local conditions. As the fundamental principle of sound practice is to use that type of road or pavement which is economical and suitable for a given set of conditions, it would be of great value to American engineers to have at hand especially more detailed knowledge relative to the traffic on the various types of roads referred to in the paper.

Another part of the paper which it is believed could be advantageously amplified is that relative to the effect of different percentages of sand-clay mixtures retained on the 10-mesh sieve, when such mixtures are used under various climatic and traffic conditions.

There are many highway engineers who, without doubt, will disagree with the author in dividing gravel and sand on the basis of material retained on or passing the 10-mesh sieve. The speaker believes that the 4-mesh sieve is rapidly being adopted as the basis of division between these materials.

Mr. Stewart.

Spencer J. Stewart, Assoc. M. Am. Soc. C. E.—The speaker has read this paper with great interest. From it it is evident that the clay-bearing materials are being given proper economic consideration in the highway development of the South. In New York State, however, clay is a material which, according to many highway engineers, possesses little or no value, but, on the contrary, contributes characteristics highly detrimental to proper highway construction.

Specifications prepared by highway commissioners, especially those of the State of New York, have insisted that the foundation course shall be filled with screenings, gravel, or sand, and have invariably ruled against a filler which would be classified as clay. From the speaker's experience, covering responsible charge of the construction of more than 300 miles of highways, at a total cost of $4 000 000, he has been led to believe that a clay-bearing material, if applied as a filler when dry, makes a foundation as stable, and less likely to disintegrate, than a filler of sand or the screenings of a non-cementitious stone.

It is common knowledge that clay may be used to advantage in binding the top course of macadam roads in cases where the dust from the crushed stone possesses no cementitious quality.

Perhaps it is not inappropriate, and may prove interesting, to call attention to the results obtained from the use of the sand-clay-bearing gravels of the Hudson River as a material for successful road construction. What is true of the cementitious gravel of the Hudson would be true of similar deposits in other parts of the country, but it should not be presumed that every gravel bank necessarily contains the characteristics of a cementitious gravel.

This material has been used for years on the Parkway Systems of Greater New York, originally as a water-bound material and, more recently, covered with hot oil. The roadways proved very satisfactory until the advent of the motor bus, which unusual traffic they could not withstand, nor was it ever intended that they should.

This same material has been used on the sandy soil of Long Island with excellent results, in spite of the crude method, or rather lack of method, used in the construction of some of the highways. Frequently, it was dumped on the sandy road, spread carelessly, and left for the traffic to compact into a smooth surface. In spite of this, however, it soon ironed out and made a most excellent road covering, considering the time and money spent on construction. On the other hand, under the direction of the New York State Highways Commission, roads of the highest type have been built of this material by the so-called water-bound method and the application of hot oil, proper care being exercised as to rolling and puddling.

With respect to roads of this latter class, the speaker wishes to call attention to one partial failure, point out the causes, and perhaps draw a lesson from the experience.

Hot oil had been applied during the late fall, and properly covered, but, in the following spring, the heat of the sun caused the oil to bleed and adhere to the iron tires of slow-moving vehicles, which drew up part of the top course and deposited it a few feet ahead. This resulted in alternate holes and mounds, and caused a very uneven surface which gradually became more pronounced as the holes became larger through wear. As soon as the first indication of bleeding occurred, the surface should have been covered with sand to prevent this unfortunate condition. One enterprising citizen did apply this remedy in front of his own property, with the result that the top surface is as good to-day as it was at the time of the completion of the road. Mr. Stewart.

Hot oil treatment on water-bound roads, whether of stone or gravel, must have a certain amount of immediate attention during the period when the oil is susceptible to bleeding. Such attention simply means the application of blotting material.

As a foundation course on sandy soils, cementitious gravel has proved an unqualified success, at least in the speaker's experience, for he has known of no failures. Failures do not mean that a stretch of 50 ft. in 10 miles might not prove unsatisfactory; that might happen with any foundation, due to causes which could not be guarded against at the time by the construction engineer. The larger the mineral aggregate, the better the foundation, if just sufficient sand and clay are used to fill the interstices. If a mixing-method top is to be placed, the speaker would advise that the foundation course be thoroughly puddled and dried before the top course is laid.

As a foundation course for country highways, it is believed that gravel containing fines of a cementitious nature is to be preferred to either broken stone or concrete.

First, it costs less. Incidentally, it might be of interest to know that practically all materials entering into the construction of roads on Long Island must be imported, whether it be crushed stone, Hudson River gravel, or so-called Long Island gravel. The latter exists only on the north shore of the Island in any quantity sufficient to be used economically for work of any moment, must be dredged, barged to Long Island City, there transferred to cars, and then hauled to its destination. Even then, this material should only be used in concrete construction.

Where materials have to be imported, it should be remembered that broken stone, even at the same cost at the point of destination, costs approximately one-third more than gravel, due to its consolidation of about 33% under the weight of the roller, and also to the necessity of adding from 25 to 30% of sand or screenings to fill the voids. On the other hand, gravel, when measured in bulk on barges or cars, will about equal the quantity rolled in place on the road, requiring no extra material for filling. Concrete requires about 80% of the

Mr. Stewart. rolled quantity of stone and gravel, with the addition of 40% of sand, together with the cement, the quantity of which depends on the proportion of the mixture.

On the contrary, where the importation of road-making material is unnecessary, the use cementitious gravel saves the cost of crushing and the incidental expenses associated with such process.

Second, a proper gravel foundation possesses the resiliency which is generally recognized as necessary for a successful road, especially for country highways. Broken-stone foundations possess this characteristic in a less degree, and concrete is entirely lacking in it.

In the case of asphalt pavements, they are now designed to overcome the lack of resiliency of the concrete foundation by the use of an intermediate or cushion course between the concrete and the asphalt surface. In the same manner properly designed mixing-method pavements of small mineral aggregate when placed on concrete foundations, should be provided with this cushion course.

Where conditions of traffic require a pavement of considerable permanency, the mixing method may be used. The speaker designed a pavement consisting of a mixture of asphalt and gravel in the proportion of 1 cu. yd. of loose gravel to an average of 20 gal. of asphalt, the gravel containing not less than 10% of clay. The gravel was bank run, the largest particles of which were 2 in. in the longest dimension, containing sufficient fines to fill the voids partly. The bitumen was a fluxed natural asphalt with a penetration between 10 and 13 mm. when tested for 5 seconds at 77° Fahr., on a No. 2 needle weighing 10 grammes.

The gravel was heated in a mechanical revolving dryer with a temperature of more than 225° Fahr., after which the asphalt, heated to not less than 275° Fahr., was added; then the mixture was placed in a revolving mixer until thoroughly and completely coated with bitumen. The mixture, at not less than 225° Fahr., was spread on the prepared bottom course with shovels from dumping boards and raked to a uniform surface with hot rakes, after which it was rolled with a self-propelled roller weighing at least 10 tons, until it was thoroughly consolidated.

On some sections of this pavement there was placed ½ in. of gravel screenings containing not less than 10% of clay which was saturated with water and rolled thoroughly and continuously until a clay mortar had been obtained. This process filled all the surface interstices with a gritty and adhesive substance which made the road practically "non-skid". In a short time the traffic drove away all surplus screenings, leaving a mosaic surface.

A different method of treatment was applied to this form of pavement in the more thickly settled communities. After the mixture had been rolled, it was covered with a coat of hot oil as a seal coat,

this seal coat being applied in November. In the following spring, under the action of the sun, the road bled to some extent, and became so sticky in a few places that the oil adhered to wagon wheels, which pulled up some of the top course. This condition could have been avoided if the authorities in charge had covered the pavement with sand or gravel screenings as soon as it became apparent that the hot oil had a tendency to bleed. No material harm was done, however, as the continuous traffic carried sufficient sand and dirt on the pavement so that in a short time the stickiness of the oil had disappeared. Mr. Stewart.

Such treatment obviates the general complaint against the so-called stone-mixing method pavement where a greater quantity of asphalt is used and where the seal coat is of the same consistency as the asphalt binder in the top course proper. These objections arise from the hardness and slipperiness of the surface, which, during the greater portion of the year, make it undesirable for horse traffic because of the former characteristic and to motor traffic on account of the latter.

It is suggested that, where the hot oil is omitted at the time of the original construction, the mosaic surface be covered with a hot oil treatment of ½ gal. per sq. yd. during the following year. With the hot oil as a squeegee course, in place of the seal coat of the same consistency of the bitumen, the pavement proper gives a less hard and slippery surface, which is desirable on country highways.

The speaker's experience has convinced him that a large plant, costing from \$5 000 to \$8 000, is not necessary in order to construct this pavement successfully, and that equally good results can be obtained from the use of small mixers costing from \$1 500 to \$2 000.

This pavement without the oil treatment cost about 85 cents per sq. yd., and with an oil seal coat about 90 cents per sq. yd. for a pavement 2½ in. in depth. This cost compares favorably with similar figures for mixing-method pavements in other parts of New York State. The speaker quotes from a statement attributed to a superintendent of highways, of New York State, relative to costs of similar pavements of graded stone material, as follows:

1½ in., California asphalt	\$1.20	per sq. yd.
2 in., Topeka	1.20	"
2 in., Warrenite	1.30	"
2 in., Bitulithic	1.60	"

The gravel used in this pavement cost approximately \$2.35 per cu. yd., f. o. b. destination, but where material can be obtained near the site of the works, the cost of 85 cents per sq. yd. can be materially reduced.

From observations of mixing-method pavements laid in New York City and vicinity, with graded stone as a mineral aggregate, it is ventured that the percentage of disintegration is as great, if not greater, than in the bituminous gravel pavement laid during the same

Mr. Stewart. period of time. In fact, after 1 year's wear, out of a total of 112 000 sq. yd., or 0.04 of 1%, less than 45 sq. yd. of this gravel mixing-method pavement had disintegrated.

The road which received the treatment of hot oil in a squeegee course was dug up in many places during the year following its completion for investigating purposes, and, where removed, the material thrown back retained its vitality to such an extent that the top course healed itself so that a casual observer could not discover where the pavement had been disturbed.

In designing a country highway, not only the original outlay, but the cost and ease of maintenance should be given proper consideration. Those who have had experience in the upkeep of gravel roads realize the small expense necessary to retain them in their original condition. Where slight depressions occur from time to time, all that is necessary is to place a sufficient quantity of gravel in these depressions and allow the traffic to consolidate it. In cases where the road has become pitted to such an extent that it requires more extensive treatment, it can be scarified, harrowed, re-rolled, and puddled, and, with the addition of a small quantity of material, can be made as good as the original at a comparatively slight cost, due to the presence of the clay-binding material. This treatment cannot be successfully consummated in broken-stone roads, for as soon as the bond of an old stone macadam road is broken, the material is usually worthless. These peculiarities of the two materials result in a very pronounced saving in the maintenance of gravel roads, as compared with that of broken-stone roads, whether or not they had been subjected to hot oil treatment. The ease of maintenance of the gravel, mixing-method pavement is sufficiently illustrated by the self-healing incident referred to.

The speaker is led to believe that gravel, sand, and clay have not been given the consideration they deserve by the highway engineers of the United States. In a section of the country which possesses clay and sand-bearing gravel, engineers can well afford to investigate its uses for road-making purposes, either as a foundation course, a water-bound top course with or without hot oil treatment, or a top course using the mixing method.

To conclude, the speaker cannot express himself more clearly, relative to the use of gravel, than by referring to the following statement:*

"A well built and well maintained gravel road is preferred by automobilists to macadam roads or even roads paved with more expensive materials. The gravel road is springy and resilient and gives good grip to the tires. It does not produce the jar that exists in riding over a paved road with an unyielding foundation."

This is also true of the man who uses his horse for either business or pleasure.

* From *Engineering News*.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

REPORT ON A SERIES OF TESTS ON CONCRETE COLUMNS REINFORCED WITH A SPIRAL OF STEEL.

Discussion.*

By Messrs. Edward Godfrey, and A. W. Buel.

Edward Godfrey, M. Am. Soc. C. E. (by letter).—This is a valuable series of tests. It is unfortunate that such information, and proper interpretation and digestion of the same, were not available before common practice in reinforced concrete design was crystallized into the deplorable shape that it has taken in all building codes and in the report of the Joint Committee on Concrete and Reinforced Concrete, as well as in the standard books on the subject. Mr. Godfrey.

"Some experiments then made on the ultimate strength of columns with longitudinal steel bars embedded in the concrete indicated that such reinforcement, unless hooped or banded at frequent intervals, could not be relied on to increase the strength of the column, and in some cases might even render its strength less."

It is gratifying to know that truths, the recognition of which the writer has been seeking since 1906, are beginning to dawn on the Profession. He predicted that it would take 10 years. It will probably take that before the Joint Committee and the authors of building codes recognize the truth. Books will follow some years later, and the rodded column will have eked out its murderous existence.

The writer has followed this subject pretty closely, and, so far as he is able to learn, no one but himself had condemned the rodded column until recently, when it was criticized severely at a meeting of an engineering society in Chicago. It is absolutely true, as the

* This discussion (of the paper by C. G. Wrentmore, M. Am. Soc. C. E., and Messrs. Hugh Brodie and C. O. Carey, published in February, 1914, *Proceedings*, and presented at the meeting of March 18th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Godfrey.

authors state, that years ago experiments indicated that longitudinal rods do not reinforce a column, but it is just as true that the very authorities who made these experiments and who discussed them in their books, endeavored to explain away the facts and to apologize for the particular specimens of concrete in the weakened so-called reinforced columns. These same authorities recommend the addition of from 15 to 39% of strength for every 1% of this weakening steel.

What are the facts to-day? Every building code allows rodded columns; practically all the standard books recommend them; the Joint Committee Report allows them; nearly every great wreck has had them, and has been the result of them.

The authors are wise to ignore rodded columns in their tests and investigation, for the rodded column has absolutely nothing to recommend it to any intelligent man, and there can be no rational incentive to make tests on them except for the purpose of demonstrating their unfitness for structures, the unfitness that an intelligent analysis of the column will demonstrate, entirely apart from tests. It is deplorable that, though the Profession has been in possession of this experimental demonstration for a number of years, the facts have been so perverted, and recommendations have been so utterly contrary to what they have shown, that standard methods of constructing columns are criminally bad.

The writer would put quite a different interpretation on the tests made by the authors from that given in their paper. Their comparison between the plain and the hooped column is unfair to the former. Not that the plain column has any right to consideration as a structural member, but such comparisons give the hooped column an apparent advantage over other structural members, in the matter of unit stress, which it does not possess.

In calculating the unit load at first sign of failure, they have taken only the area of the core for columns with hooping, whereas they take the full sectional area of the plain columns. The area of the core is about 10 sq. in. and that of the full section is about 13 sq. in. Making this approximate correction on all their columns having embedded coils, the writer has discovered the following remarkable facts: The average load at first sign of failure for the 24 plain columns is 2 250 lb. per sq. in.; the average load at first sign of failure for the 18 columns having less than 1% of hooping is 2 250 lb. per sq. in.; the average load at first sign of failure for the 28 columns having more than 1% of hooping is 2 230 lb. per sq. in.; and the corresponding average values for the ten highest in the foregoing groups are:

Plain columns, 3 057 lb. per sq. in.
Less than 1% hooping, 2 614 lb. per sq. in.
More than 1% hooping, 2 769 lb. per sq. in.

Mr. Godfrey.

If these facts teach anything at all, they blazon out the truth that hooping has no influence whatever on the first failure of a column, and it is the first failure that is really the ultimate failure. A column is useless after it begins to spall on the surface.

The data presented by the authors on the ultimate strength of these columns are only of academic interest. They are totally erratic, and simply represent the post-mortem strength of columns which have already failed. How long it may take a rattlesnake's tail to cease wagging after its head is crushed, has no bearing on the safety of the persons who were menaced by the live snake. The load-resisting power of a more or less disintegrated mass of concrete encircled by a steel spiral has no bearing on the design of a safe structure, if the first sign of failure is ignored. The authors might as well take one series of columns and label them "A" and use their full area in determining the unit load, and then take another set which have gone through some treatment or other and label them "B", and use only 80% of their area, and then say, when Columns "B" stood exactly the same load as Columns "A", that Columns "B" were 25% better than Columns "A", as to make the comparison that Table 3 shows on its face.

We have it on the authority of Ernest McCullough, M. Am. Soc. C. E., that in a large number of commercial designs he has never found one which excluded the outer shell in the strength of the column. The writer has persistently refused to have anything whatever to do with the checking of rodded column installations, or doubtless he would have discovered the same thing. There is no reason why investigators should omit the outer shell of a column, either in making comparisons or in discovering the strength of columns.

These tests demonstrate several things. One is that the closer the pitch of the spirals, in general, the greater the ultimate strength of the column. This could be reasoned out by analogy. A brick will stand more load per square inch flatwise than on end. A mortar joint is stronger the thinner it is made. A flat disk will stand much more load than a high cylinder of the same diameter. There is a limit, however, beyond which this added strength is of no use. It would be unsafe to have a column in a structure with the shell outside of the coil spalled off. Hence the "first sign of failure" is the function that must govern in safe design. This value, in these tests, runs from 1 400 lb. per sq. in. up, on the full area in compression, on the hooped columns, or 1 800 lb. on the core.

Now what factor of safety will the Pittsburgh Building Code give, for example, allowing as it does from 1 300 to 1 750 lb. per sq. in. of alleged safe load on columns, as shown by T. L. Condron, M. Am. Soc. C. E..* Other building codes are not quite as bad as that of

*Paper before the Western Society of Engineers, Feb. 9th, 1914; *Engineering News*, Feb. 19th, 1914; *Engineering Record*, Feb. 21st, 1914; *Engineering and Contracting*, Feb. 18th, 1914.

Mr. Godfrey. Pittsburgh, but those of St. Louis and Minneapolis are close seconds. The gravel concrete common in Pittsburgh is not nearly so strong in cylinders as the limestone concrete used by the authors.

In a testing machine, a column might show a certain strength at first failure, and two or three times as much at ultimate failure, but, if it begins to fail in a building, the sway and eccentricities unavoidable in structures will doubtless mean its ultimate failure.

One of the purposes of a factor of safety is to cover imperfections. What would happen in this "safe" Pittsburgh column, if a workman should drop a wooden block into it accidentally, or if there should be a void, filled up later by a trowel full of mortar? Who would be responsible for the possible general wreck of the building, the workman, the designer, or the commercial interests who get up these building codes? Laboratory tests are more carefully made than actual construction, so that this should be taken into account.

These tests demonstrate that there is no definite relation between the percentage of steel reinforcement and even the ultimate strength of the core. This, together with the facts concerning the pitch of the coil, shows clearly, what the writer has for many years contended, that Considère's formula is wrong. That formula, and it is used in the newest building code, which is that of Pittsburgh, would add to the strength of a column having a given pitch by adding to the size of the wire in the pitch, a perfectly absurd proposition, as the wire might already be strong enough to stand the crushing strength of the disk between two coils.

Another important fact brought out by the authors is that the ratio between the moduli of elasticity of steel and concrete is not more than 10. The value, 15, is very commonly used with a resultant degradation of design, where steel rods are considered in compression.

The authors state:

"That the shrinkage of the concrete, causing initial compression in the steel, might be the cause [of greater deformation in hooped columns] is untenable, because the concrete was immersed in water during the entire aging period."

It is presumed that the columns were dry when tested. Tests show that specimens shrink and swell by the mere process of drying and soaking them.

What the Profession needs is to ascertain the combination of spiral and longitudinal reinforcement for a column of given diameter, which will be consistent and properly balanced. This should then be adopted as a standard, and the value of a column should be based on the kind of concrete used, just as in beams or slabs. The longitudinal steel should be stiff enough and the coils should be close enough to hold these rods from buckling. The writer* advocated this in 1906 and worked out such a standard.

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*In his book entitled "Concrete."

The Profession is not in need of tests so badly as of proper interpretation of those already made. This is not said in disparagement of those made by the authors. They are of great value. What is sorely needed is complete revision of building codes, books, and of the Joint Committee Report. The present standards of reinforced concrete design are dangerous and outrageous. Mr. Godfrey.

A. W. Buel, M. Am. Soc. C. E.—It appears to the speaker that the programme for this series of tests was planned so that results or conclusions of much practical value or reliability could hardly be expected, at least none commensurate with the expenditure. Mr. Buel.

The slenderness ratio, about 6.75 for the concrete or 7.7 for the coil, was much too small to represent conditions in practice, and should classify the series with compression tests rather than with column tests.

The concrete mixture, 1 : 2 : 4, was too rich to use economically with spiral or hoop reinforcement, and the tests do not show the true value of such reinforcement, which is greater with lean concretes. Some indication of this is shown by a comparison of the short- and long-time tests of this series.

The section of the test columns was so small that the results will hardly be depended on as a basis for structural designing, in view of the fact that engineers already have reports of a considerable number of full-sized tests. Some 6 or 8 years ago, when very few reports of full-sized tests were available, such tests as these would have been received as a valuable contribution. A dozen full-sized column tests, carried out on a well-planned programme, would be a more valuable addition to the experimental data on the subject than several hundred tests on small model specimens having dimensions entirely out of proportion to those generally found in practice.

The pitch of the spirals varied from 0.25 to 0.75 in., in columns of the same group. This, and also the variations in age, make it impossible to compare fairly the results from one test column with those from another, to draw reliable conclusions, or, for practical purposes, to realize what the authors describe as the "Aim of the Tests".

The records of "first sign of failure", particularly for the short-time tests, do not agree with tests reported by other experimenters, where the envelope of concrete or cement outside of the spiral or hooping almost invariably began to show signs of failure as soon as the load reached or exceeded the ultimate load of a similar column without reinforcement. In other words, all previous tests have indicated that spiral or hoop reinforcement does not materially raise the point of first sign of failure. Mr. Godfrey has pointed out what seems to be an error in computing the intensity of stress at first sign of failure: that the area of cross-section enclosed by the spiral was used, whereas the area of the entire column, including the concrete outside of the spiral, should have been taken for this computation. For the determin-

Mr. Buel. ation of the intensity at the ultimate load, the area enclosed by the spiral is correct, because at that time the outside envelope would have been destroyed.

In a number of cases the pitch of the spirals was somewhat higher than that recommended by Considère for maximum efficiency of hooping. This was confined mostly to columns with the smaller percentages of spiral reinforcement.

Most of the eight conclusions drawn by the authors from the results of this series of tests are in substantial agreement with those deduced from previous experiments, and can be considered as a confirmation of the latter.

Conclusion No. 2 gives a value for e, the ratio of the modulus of elasticity of the steel, E_s, to that of the concrete, E_c, which is remarkably low, considering what engineers have been accustomed to use and the results of previous experiments. The speaker would like to ask the authors whether the value of E_c, which they derive from the formula, $600\,000\ f_c \div s_2$, is the true value of E for concrete alone, and is it not possible that it was affected by the spiral reinforcement to some extent? Also, what was the value of E_c for the plain concrete columns without reinforcement? Is not the E_c computed by the authors' formula, the E for the column as a whole, rather than the E for the plain concrete?

Conclusion No. 5, that the spiral reinforcement materially raises the limit of the first sign of failure, will require further experimental demonstration to be fully convincing, in view of the considerable number of determinations which have shown different results.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

Discussion.*

By Messrs. Philip W. Henry and James J. Hill.

Philip W. Henry, M. Am. Soc. C. E.—This paper contains a great deal of valuable information, particularly for those interested in the construction and operation of railways in Latin-America, and the author is to be congratulated on the exhaustive and intelligent manner in which he has treated this important subject. Of special value is his emphasis on the distinction between a "narrow-gauge railway" and a "light railway", which are often used as synonymous terms, although, as the author points out, a "light railway" may be of any gauge. Mr. Henry.

The speaker agrees with Mr. Lavis that the difference in the construction cost between a narrow-gauge and a standard-gauge railway is generally exaggerated. In a country the traffic of which does not admit of the construction of an expensive railway, and where the prevailing gauge is 4 ft. 8½ in. or greater, the speaker believes that it is better to cut the construction cost by raising the maximum grade and curvature rather than by lessening the gauge of the railway. Though the author treats of railways chiefly in the southern part of South America, more particularly Argentina and Southern Brazil, both lying within the temperate zone, where conditions are quite similar to those in the Central Western States, and where traffic will follow railway construction almost as rapidly as it did in those States, it must be remembered that in tropical South America and in Central America traffic does not follow with the same rapidity, and, therefore, the type of railways for those countries may be quite different from that suitable to Argentina and Southern Brazil. Broadly speaking, it is safe to

*This discussion (of the paper by F. Lavis, M. Am. Soc. C. E., published in March, 1914, *Proceedings*, and presented at the meeting of April 1st, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Henry. assume that in Central America, the West Indies, and in tropical South America, the gross earnings of a new railway for the first few years will not be much greater than operating expenses, let alone interest on the capital invested, and that the development of traffic will be very slow, unless the parties building the railway undertake at the same time to develop traffic. Even in a highly fertile tropical country, it is seldom that the natives, who are well satisfied with their existing life, will make any effort to develop traffic on any considerable scale. The development of tropical agricultural traffic in most cases demands both capital and skill. This applies particularly to the raising of bananas and sugar cane, two of the most profitable tropical products.

If the railway is built in the high altitudes of Peru or Bolivia, not susceptible of agricultural development, it is essential that mineral traffic be developed, which, again, requires large investment of capital. In some cases, the building of a railway may be followed by the investment, on the part of outside capitalists, in the agricultural or mineral resources of the region traversed by the railway; but, generally speaking, if the parties building the railway are also its operators, it is essential for best results that, at the same time they acquire the concession for building the railway, they should take steps for the development of traffic—a point which is generally overlooked. The Governments often recognize this slow development of traffic, either by guaranteeing the bonds of the railway or by paying a cash subsidy approximating, if not equalling, the cost of construction.

The speaker believes that the author has over-estimated, if anything, the additional cost of constructing a standard-gauge railway (4 ft. 8½-in.) over that of a meter-gauge, especially where he adds an extra cost for "Contingencies, Expenses of Promotion, and Interest during Construction," amounting in all to 15%, which, according to his figures, equals $249 per mile on roads of light earthwork and bridging, $430 per mile on roads of medium earthwork and bridging, and $1 359 per mile on roads of heavy earthwork and bridging. It appears to the speaker that this item of general expense is the same, regardless of the gauge of the railway. In other respects, the speaker believes that the author's estimates are, if anything, more than ample. The extra cost of standard-gauge, given by him in detail for the three classes of railways, neglecting the 15% for general items as above, is as shown in Table 28.

To confirm these figures, it is interesting to compare the actual cost of building a railway of light earthwork and bridging in Bolivia, from Viacha to Oruro, 127 miles in length, with which the speaker was connected in 1906-09. This railway was of meter-gauge, roadbed of 5 m., the average cross-section for the entire length showing excavation actually paid for, both in cuts and borrow-pits, amounting to an average depth of 3.7 ft. Assuming that the railway had been of standard-

gauge, instead of meter-gauge, it would have been necessary to handle $6\frac{1}{2}\%$ additional earthwork. It is assumed that the cost of bridges, trestles, and culverts, and of tracklaying and surfacing, would also have been increased by $6\frac{1}{2}\%$, certainly a liberal estimate. The ties, California redwood, costing about \$2, delivered on the work, were 7 ft. in length, instead of 8 ft., if standard-gauge had been adopted, making an additional cost of $14\frac{2}{7}\%$ for ties. Table 29 shows the actual cost per mile of the different items affected—in the speaker's opinion—by the change in gauge, and the additional cost for each item in case standard-gauge had been adopted. Mr. Henry.

TABLE 28.

	Light earthwork and bridging.	Medium earthwork and bridging.	Heavy earthwork and bridging.
Earthwork	\$310	\$980	\$5 010
Bridges	150	500	2 000
Culverts	150	300	1 000
Ties	770	770	770
Laying and surfacing	100	100	100
Yards and sidings	170	170	170
Cattle guards, fences, etc	10	10	10
Total for construction	\$1 660	\$2 830	\$9 060
Less saving in equipment	100	100	100
Net Total	\$1 560	\$2 730	\$8 960

TABLE 29.

	Actual cost, meter-gauge.	Estimated additional cost, standard gauge.	Total estimated cost, standard-gauge.
Grade	\$4 200	\$272	\$4 472
Bridges, trestles and culverts	2 600	168	2 768
Tracklaying and surfacing	1 600	97	1 697
Ties	5 100	737	5 837
Total for these four items	\$13 500	\$1 274	\$14 774

This shows an estimated additional cost of standard- over meter-gauge for a railway of light earthwork and bridging of \$1 274, as compared with the author's estimate of \$1 560, neglecting general expenses.

The question of gauge, except perhaps in Argentina and Southern Brazil, is not likely to arise on railways of medium or heavy earthwork and bridging, but it is more likely to arise on those railways built in districts of light traffic, where it is necessary to cut down the expense of construction to the lowest possible limit. As stated before, the speaker prefers to do this by building a railway of the gauge standard

Mr. Henry. to the district, high maximum grades and curvatures, say, $3\frac{1}{2}$ or 4% and 25° curves, intelligent discrimination being used in the application of these maxima. By using the Shay or a similar type of locomotive, the maximum grade may be run up to 8 per cent. As one train each way per day—sometimes only two or three trains per week—will take care of all the traffic, it is evident that the cost of conducting transportation is a small item when compared with the cost of maintenance of way and with the interest on the cost of construction. Therefore, under such conditions of light initial traffic and slow development of traffic, the first cost of construction, with due regard to maintenance of way, should be the controlling factor. At first sight, this would indicate a narrow-gauge railway, which admittedly can be built at from \$1 000 to \$1 500 per mile lower cost than standard-gauge, the interest on which at 7% is from \$70 to \$105 per mile per year, quite an item for a railway the gross earnings of which may run from \$1 000 to \$1 500 per mile per year. Still, other considerations must be taken into account, such as gauge of connecting railways or of railways likely to be constructed. In Bolivia the gauge of the railway from Viacha to Oruro, 127 miles, was fixed at 1 m., as it was intended eventually to connect it with the Argentine Railway on the frontier, 425 miles distant, which is of meter-gauge, thus furnishing a through route from La Paz, the capital of Bolivia, to Buenos Aires. In Haiti and Santo Domingo, where the largest mileage of railways is of 42-in. gauge, that gauge will naturally be adopted for future railways in that island. In a country, however, where standard-gauge now prevails, or is likely to prevail, the building of a narrow-gauge railway is of doubtful expediency, and the speaker would prefer reducing the first cost by high maximum grade and curvature.

Mr. Hill. JAMES J. HILL, F. AM. SOC. C. E. (by letter).—In the development of much of the territory of the United States, the original cost of construction of the 4 ft. $8\frac{1}{2}$-in. gauge was all that (and in many cases more than) the country could bear, though the writer has long felt that it is not certain but that, for very heavy traffic, a 5 ft. 6-in. gauge would have been better. The difference in the dead weight of the car would not be great, and the center of gravity would be nearer the rail.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON BITUMINOUS MATERIALS FOR ROAD CONSTRUCTION.*

By Clifford Richardson, M. Am. Soc. C. E.

Clifford Richardson, M. Am. Soc. C. E. (by letter).—This report rings the changes on a number of conclusions with which most experienced road builders are acquainted and with which they will agree. They can, probably, not be brought too often to the attention of those who are less well informed. In some directions, however, these conclusions will not be agreed to by all road builders of experience, nor will there be complete unanimity, as far as approval of some of the definitions proposed by the Committee is concerned. Mr. Richardson.

In discussing the mixing method the report states "Excessive sizes * * * , of the mineral particles should be avoided". It seems to the writer that this statement should be modified. The use of large-size stone, 2½ in., is not objectionable if the road crust is of sufficient thickness to permit it, and if the surface is well closed up with finer material, but it would be absurd to use such a stone in a 3-in. thickness, for instance. The size of the stone should be accommodated to the thickness of the road crust, but where large stone can be used, the surface shows greater stability with large-size particles than with small ones.

Under the same subject, the report says that "Mixing machines should be used, and hand-mixing methods should be avoided wherever practicable". The difficulty with the mixing process lies in the fact that most mechanical mixers will not handle large stone in a satisfactory manner, and that hand-mixing is necessary for such a purpose. In the writer's experience, some of the best roads with which he is acquainted have been constructed by hand-mixing processes, as, for

* Continued from February, 1914, *Proceedings*.

Mr. Richardson. example, Western Avenue, in Gloucester, Mass., which was built by this process in 1909.

All road builders of experience will agree with the Committee's conclusion that the use of fine sand on top of bituminous concrete is open to grave objection, and that clean stone chips, or small gravel free from particles that will pass a $\frac{1}{8}$-in. mesh screen, is preferable.

Among the Committee's conclusions in regard to the construction of road surfaces by the penetration method is one which states that "An important factor for successful results is the thorough compaction by rolling of the road metal before the spreading of the bituminous material". In the writer's experience, though this may be true where the bituminous binder is one which melts to a slightly viscous liquid and, in consequence, might not remain in place with the stone not thoroughly rolled, it is far from being so when the binding material consists of the more viscous and stable natural asphalts. Experience has shown that better results are obtained in using the natural asphalts for penetration purposes if the rock is merely spread, the road asphalt being applied thereto without rolling, followed by a course of smaller stone, and compaction after this application. The Committee is certainly right in stating that "the use of fine sand on top of the bituminous material is open to grave objections" and that clean stone chips are preferable. No one who has had extended experience in road construction could differ from these conclusions.

Not all engineers will agree with the Committee's suggested definition for an "aggregate", in so far as the statement goes that "the broken stone may be referred to as the aggregate and the sand, stone dust, or other fine material may be referred to as the binder". If such a definition were accepted a sheet-asphalt surface would contain no mineral aggregate.

The definition proposed by the Committee for bitumen is that adopted by the American Society for Testing Materials, as suggested by a Committee of that Society. At the meetings of that Committee and, before the Society, the writer raised very strong objections to the use of the word, bitumen, in a sense other than that of its etymological origin, and presented a Minority Report,* which, as the definition has again been proposed for presentation to the American Society of Civil Engineers, should, he thinks, be brought to the attention of the membership. It is, therefore, here reproduced:

"In the writer's opinion, the definitions presented are unsatisfactory, both in matter and form, and in one instance is quite contrary to the etymological significance of the word defined. The majority report defines bitumen as a 'mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, etc.' The Committee throws over all authorities that have heretofore ruled on the subject, and provides

* *Proceedings*, Am. Soc. for Testing Materials, Vol. XII, p. 75.

that by its arbitrary decision coal tar is a bitumen. It is stated by the Committee that it believes the definitions of the dictionaries and encyclopædias are unsatisfactory, and brushes aside the following definitions of our best known dictionaries and writers: Mr. Richardson.

" 'Bitumen: The name given by Latin writers, especially by Pliny, to various forms of hydrocarbons now included under the names asphaltum, maltha and petroleum'. "Century Dictionary, Edw. S. Dana, Ph. D.

" 'In modern scientific use, the generic name of certain mineral inflammable substances, native hydrocarbons more or less oxygenated, liquid, semi-liquid, and solid, including naphtha, petroleum, asphalt, etc.' "New English Dictionary, Murray.

" 'Any native mixture of hydrocarbons, oxygenated, as naphtha and especially asphalt'. "Standard Dictionary.

" 'By extension, any one of the natural hydrocarbons, including the hard, solid, brittle varieties called asphalt, the semi-solid maltha and mineral tars, the oily petroleums, and even the light, volatile naphthas.' "Webster's Dictionary.

" 'This term includes a considerable number of inflammable mineral substances consisting mainly of hydrocarbons. They are of various consistence, from thin fluid to solid, but the solid bitumens are for the most part liquefiable at a moderate heat. The purest kind of fluid bitumen called naphtha, or rock oil, is a colorless liquid of specific gravity 0.7 to 0.84 and with a bituminous odor. It often occurs in nature with asphalt and other solid bitumens. Petroleum is a dark-colored fluid variety containing much naphtha. Maltha or mineral tar is a more viscid variety. The solid bitumens are asphalt (q. v.) mineral tallow or hatchetin; elastic bitumen, mineral caoutchouc or elaterite; ozokerite.' "Dictionary of Applied Chemistry, Thorpe.

" 'A generic name for a variety of substances found in the earth, or exuding from it upon the surface, in the form of springs. The liquid varieties become inspissated by exposure and eventually harden into the solid form, which is asphaltum.' "Appleton's American Cyclopædia.

" 'Bitumen is the name used to denote a group of mineral substances, composed of different hydrocarbons, found widely diffused throughout the world in a variety of forms which grade from thin volatile liquids, to thick semi-fluids and solids, sometimes in a free or pure state, but more frequently intermixed with or saturating different kinds of inorganic matter.' "From 'Highway Construction', by Austin T. Byrne.

" 'The word bitumen may, therefore, be strictly defined as a general term that is used to designate a class of minerals as they occur in nature, that are soluble in chloroform and other neutral liquids.' "S. F. Peckham.

" 'Any mixture of hydrocarbons and their derivatives of mineral occurrence whether solid, liquid or gaseous, which is soluble in chloroform or similar solvents.' "George W. Tillson.

Mr. Richardson.

"It seems to me that this is a most arbitrary procedure. If the definition in Thorpe's Dictionary of Applied Chemistry cannot be accepted as a satisfactory one for bitumen, the entire structure of chemical nomenclature might be overthrown.

"As a substitute for the definition proposed by the majority of the Committee, I would suggest the following:

"Bitumen is a material found in nature, consisting of a mixture of hydrocarbons and their derivatives, which may be a gas, liquid, a viscous liquid or maltha or a solid but, if solid, melting more or less readily on the application of heat and soluble in carbon disulphide or similar solvents.

"The definition of 'bituminous' is equally unsatisfactory from the writer's point of view, being defective in so far as it is based on the previous definition of bitumen. The writer would suggest the following:

"A material is said to be bituminous when it contains bitumen or material resembling bitumen, or if it yields bituminous material, or if it constitutes the source of bituminous material. Coal-tar is called a bituminous material from its resemblance physically to some of the denser forms of the native bitumen, although it contains no bitumen. Bituminous macadam is a road surface bound with bitumen or bituminous material."

To this Minority Report, four members of the Committee replied that the application of the term bitumen to coal-tar is sanctioned by Allen in his "Organic Analysis", Dr. David T. Day of the U. S. Geological Survey, Prévost Hubbard, Assoc. Am. Soc. C. E., of the Committee, in his publications, by the American Railway Engineering Association, in the "American Civil Engineers' Pocketbook," and in the Progress Report* of the Special Committee on Bituminous Materials for Road Construction, of the American Society of Civil Engineers, and that bitumen is used, in the sense adopted by the Society, in the specifications of such States as New York, Pennsylvania, Illinois, New Jersey, etc., and by many city authorities like George W. Tillson, M. Am. Soc. C. E.

On the other hand, such an authority as M. Paul le Gavrian, Assistant Secretary, International Association of Permanent Road Congresses in Paris, has stated in reply to an inquiry from the writer:

"I am of your opinion as regards the abuse of the word 'bitumen'. It should be used only to denote natural products and not the tars and their derivatives. I go even farther than you and wish that the term 'bitumen' might be reserved solely for materials containing natural bitumen."

Again, Colonel Crompton, Consulting Engineer of the Road Board of Great Britain, writes:

"You are right in supposing that I agree with you that coal-tar is certainly not bitumen."

* Presented to the Annual Meeting of the Society, on January 17th, 1912.

The subject is to be considered by the Engineering Standards Committee of Great Britain in conjunction with some of the American authorities, in consequence of a meeting held in London at the time of the recent International Road Congress. In view of what M. le Gavrian and Colonel Crompton think, it hardly seems to the writer that there can be any unanimous acceptance of the American definition for bitumen at the present time, and, this being the case, it would seem to be wise not to attempt, in view of the controversial nature of the matter, to force it on the engineering world, by a majority vote, which will probably chiefly be cast by individuals not directly interested in the subject, even if the majority is very large. Mr. Richardson.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By Messrs. F. Lavis, H. C. Phillips, William W. Crehore, M. R. Maltbie, Henry Floy, V. K. Hendricks, William J. Wilgus, Richard T. Dana, R. S. McCormick, Richard J. McCarty, A. W. Buel, J. E. Willoughby, S. S. Roberts, C. P. Howard, James D. Mortimer, Charles S. Churchill, M. H. Brinkley, H. M. Stone, S. Whinery, Philip W. Henry, Charles Hansel, and D. W. Lum,

F. Lavis, M. Am. Soc. C. E.—This report is entitled "Report of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property," etc., with the sub-title "Valuation for the Purpose of Rate-Making." The speaker agrees that it is an important contribution to the literature of valuation; he thinks it unfortunate, however, that, at least by inference, it fosters the belief that the principles formulated and proposed can or may be used in the valuation of railways for the purposes of rate-making. Mr. Lavis.

The title and designation of the Committee, as well as the proposal for its appointment, leave no doubt that the valuation of the railways was uppermost in the minds of the Board of Direction when the Committee was appointed; it seems unfortunate, therefore, that only one railroad engineer was appointed on it.

In view of the nature of the report, the speaker is strongly of the opinion that it is unfortunate that the word railway or any reference to the railways or their rates was permitted to appear in connection with it. He suggests that either the question as affecting railways be entirely eliminated, and a clear statement made at the beginning of the report that such is the case, or that the Committee be increased by

* Continued from March, 1914, *Proceedings*.

Mr. Lavis. the addition of three or more members of wide experience and training in railroad location, construction, maintenance, and operation. Such contribution as the speaker offers in the discussion which follows, is directed more particularly to pointing out what he considers to be unwarranted inferences in regard to the valuation of railways which might be drawn from the report.

It is noted that in the report the expressions "physical valuation of property" and "valuation of physical property" are used quite indiscriminately. One of the fundamentals of the establishment of principles is the definition of terms. It is suggested that the Committee establish the use of the expression "valuation of physical property" to mean the establishment of the value of the inventoried items, without consideration of the so-called intangible values; also, that in speaking of the determination of the value of the property as a whole, that is, as a going concern, the word "valuation" or "appraisal" be used alone, without the addition of the word "physical." As an instance of the confusion of terms, it may be noted that the Committee includes "the cost of development" in what it calls the "physical value." This cost, of course, is part of the value of the property, but is hardly of the same class as the value of the rails, ties, etc., which are physical. The statement is made that physical valuation "does not furnish a satisfactory basis for rate regulation", but that other things must be considered. What is meant, presumably, is that other things than the value of the physical property must be considered. Among these items are mentioned location, design, and efficiency of operation, but it is admitted that the formulation of principles whereby values of these items may be determined for the purpose of rate regulation is somewhat difficult. Mr. Gibson in his discussion* also advocates the inclusion of such intangible items as strategic location, efficiency, etc.

The speaker has pointed out, in his discussion of the paper† by W. J. Wilgus, M. Am. Soc. C. E., the fallacy of the supposition that rate regulation of any individual railroad could be based on the result of any valuation of any kind; and his point of view and its reasonableness have been specifically and in detail publicly confirmed since then by the Hon. Charles A. Prouty, of the Interstate Commerce Commission.‡ It is out of the question, of course, to assume that the efficiency of the management of, say, one of the four through lines between New York and Chicago could affect its rates, or that its physical characteristics, as shown by its profile, alignment, or length, could do so. These things are items affecting the values of the different lines, but not their values for purposes of rate-making, or even rate regulation.

* *Proceedings*, Am. Soc. C. E., for February, 1914.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 283.

‡ An address before the U. S. Chamber of Commerce at Washington, D. C., February 11th, 1914, *Railway Age Gazette*, February 13th, 1914.

Mr. Lavis.

A most excellent exposition of the various factors governing the establishment of the rates on railroads has recently been contributed by Mr. G. C. Hand,* the details of which it is not necessary to repeat, suffice it to say that he shows clearly that railroad rates in the United States are based on such a variety of different factors, including competition of different lines between the same points, a limited maximum which can be charged to move certain business, the accessibility of markets, balance of traffic, etc., etc., that neither the value nor the cost of service, nor the value of the property involved, are more than one of several factors—and often subordinate ones—in the determination of the rates on any individual railroad.

The Committee has quite wisely taken the position that valuation—of railroads or other property, as a whole and as a going concern—may, and should, be governed by the purposes for which it is made, and realizing the vastness of the subject decided to limit the scope of its inquiry. The speaker ventures to suggest, however, that the subject might well have been, and still might be, narrowed down to the consideration of the question as affecting the determination of the values of the physical property of railroads, or that the railroads be left to their own resources in working out their own salvation.

It is perhaps pertinent to the discussion to point out that though the valuation of the physical property of the railroads is very largely an engineering work, the valuation of the whole property of our railroads can only be properly made by a combination of financial, legal, and operating experience, combined with engineering; and that any committee of this Society would do well to hesitate before formulating principles for the determination of values other than of the physical property.

The Committee has devoted 18 pages of its report to the subject of depreciation, and this has been followed by a further supplementary statement of 18 pages on the same subject, and certain tables have been presented in the addition to the report. Appreciation is disposed of in 2 pages, and its discussion is practically confined to the consideration of increase in land values. The speaker, in his discussion of Mr. Alvord's paper,† expressed the belief that the physical property of any of our first-class trunk-line railways is worth to-day as much as, if not more than, its cost of reproduction new. Mr. Wilgus has expressed a similar belief in his discussion of this report, and it is undoubtedly held by every railroad man of experience, yet we are asked to believe, not only that valuations showing the present fair value of our railroads as only 85% or 90% of the cost of reproduction new are correct and just, but that rates should be based on these values.

The speaker believes that depreciation must be deducted in order

* *Railway Age Gazette*, February 27th, 1914.

† *Proceedings*, Am. Soc. C. E., for January, 1914, p. 191.

Mr. Lavis. to obtain the true value of the physical property, but he also urges a more careful consideration of appreciation, and allowance for omissions in the inventory, which he believes will show the final value of the physical property alone of an established and well-maintained railroad to be at least 100% of its cost of reproduction new.

As regards the railroads which are not efficiently maintained, a decrease of rates on account of decreased value of the property will not build them up.

This, however, is aside from the main contention of the Committee's report, which deals with the methods of determining an annual charge as a proper allowance for rate-making. So far as the railways are concerned, a much more correct expression would have been renewal charge, or allowance for renewals.

The very clear and illuminating statement* of Mr. Jared How, as to the contractual obligation of the railroads to keep their property at 100% efficiency, and the difference between the continuity of railway maintenance, as contrasted with the life of a pumping plant, for instance, is of much interest.

The determination of an annual charge for renewals on a railroad seems to the speaker to be taken care of quite satisfactorily by an annual charge of a certain percentage of the value of the property based on its estimated life, and there seems to be no use in introducing or attempting to introduce great refinements in the estimate of what this amount should be. The intricate calculation of sinking funds, compound interest, etc., together with an elaborate adjustment of the value to be allowed for the actual depreciation, the latter varying constantly with the assumed fair return and assumed life, does not seem applicable to railroads.

The property of a railroad must be considered as starting with a value of 100, reaching a certain normal state, and remaining constantly in that condition, being maintained there by yearly appropriations to cover maintenance and renewals. Theoretically, these appropriations would be nearly the same each year, but it is easily understood that they may well fluctuate slightly, and it is desirable that they should be able to do so.

There is really no question of compound interest, or sinking fund, or anything of the kind; there is a continuous yearly appropriation to keep the property up to standard efficiency.

The theory of the Committee, however, is based on property which starts at 100, decreases to zero, returns to 100, and so on, and where it is necessary to have set aside—no matter how the money may be kept or invested—a sum for this complete renewal or replacement. At first thought, it would seem as if this was true of any property, but in its application to a railroad, it seems to the speaker that this question of

* *Proceedings*, Am. Soc. C. E., for February, 1914.

depreciation is so mixed up with and related to maintenance that it can only be taken care of or included in and with maintenance expenses, or virtually as an operating cost, and this is where the railroad situation differs materially from that of other types of public utilities. For instance, the cost of renewal of rails and ties is closely correlated to and dependent on not only the quantity and kind of ballast in use, the quantity of track work performed, the climatic conditions, weather, etc., but also on the type and condition of the rolling stock and the amount of traffic. Mr. Lavis.

It may seem to be theoretically possible to take a structure like a large bridge, a great terminal, or similar large item, and calculate a sum to be set aside yearly to replace it, but really the very uncertainty of the length of their life makes this utterly impractical in application.

In a general way, it is known approximately what percentage of the total value of the physical property of railroads should be set aside yearly for maintenance and renewals, and, for all practical purposes, it seems to the speaker that this percentage, based on the experience of many years, is as near as we can come to fixing a fair value for this allowance.

If it is necessary to go into further detail, each item composing the railroad may be tabulated, its life approximated, a value fixed by deducting the scrap value from the cost new, and the yearly charge determined by dividing this value by the estimated life. The bases of these calculations, however, as will be readily seen, are so indefinite and subject so much to judgment and individual experience, that greater refinement than this seems useless, and, as in the case of railroads, this money is actually—no matter what the theoretical case may be—used up each year, it is no use to establish elaborate methods of providing for compound interest, etc.

The Committee has advanced the *reductio ad absurdum* argument, that certain methods of estimating this percentage would mean that the property owners would have, at the end of, say, 10 years, $250 worth of property for every $100 invested. As a practical proposition, however, there seems to be little danger of this, so far as the railroads are concerned.

There is little need to fear, as Mr. Bates* seems to, that fair and adequate regulation of the railroads by the Government will result in the spoliation of the widows and orphans. In spite of the general efficiency of the management of our railroads and the integrity of their officials as a whole, there are not lacking examples of some neglect by them of the interests of the large body of stockholders whose holdings are so small as to deny them effective representation. The speaker has had some experience with regulation in other countries, and he is in-

* *Proceedings*, Am. Soc. C. E., for February, 1914.

Mr. Lavis. clined to believe that the concentration of the regulation of the railways in one national body composed of men not less just and fair than the best in the country, will be a benefit rather than a misfortune, and in this he believes most of the railway officials agree. It is really a national misfortune, that adequate regulation by the National Government was not undertaken from the beginning.

The speaker believes that the valuation of the railroads which has been undertaken by the Government, if it be properly made—as with the co-operation which the railroads are giving, it undoubtedly will be—will constitute a long step forward in the acquisition of that definite, full, and accurate knowledge which is necessary for a clear understanding of the situation, which knowledge is not now possessed by any one, neither the Government, the general public, the stockholders, nor even many of the officials and directors of the railways themselves.

How this knowledge is to be applied to regulation is not clearly seen by any one just now, but the study and the knowledge obtained will undoubtedly help to a clearer view. It seems that it can only be applied to rate regulation in the most general way, and, therefore, as if the work of the Committee might well be directed to the determination of the principles governing the valuation of the physical property alone, this inventory and valuation to be made in such a way that it may be used as a basis for any of the purposes to which valuation may be applied.

Mr. Phillips. H. C. Phillips, M. Am. Soc. C. E.—The speaker regards it as exceedingly unfortunate that the Special Committee assigned to prepare a report formulating the "principles and methods for the valuation of railroad property and other public utilities", after more than a year of silence, and with no inkling of the change of direction of its labors at the Annual Meeting of the Society on January 15th, 1913, should have so limited the subject as to bring into it conditions which, as far as railways are concerned, tend to weaken the whole discussion.

With regard to the report as applied to public utilities which by their nature are somewhat in the class of protected monopolies, or, at any rate, have a well-defined sphere of action, the speaker has no criticism to offer. However, the Committee on the title-page of its report indicates that its subject is "Valuation for the Purpose of Rate-Making", and, further, that railroad properties are included throughout its report. With the exception of trolley lines, rather free from competition and more or less local in their nature, the speaker, in a study of valuation subjects extending over the past two years, has been unable to find any place where valuation has been used in the determination of rates on railroad properties, and though it is entirely true that valuation has been held by the Courts to be the test as against confiscation by rates made on altogether different principles, the

rates of a railroad have been, and of necessity must be, based on many other factors, and valuation carries a very small and insignificant part in the determination of such rates. Mr. Phillips.

To members of the Society having had railroad experience, it is hardly necessary to point out that the factor of competition as between railroads, places, and commodities, and, to a lesser extent, the cost of performing the service, have been and will be the leading factors in the determination of railway rates, and the only point at which valuation becomes a live issue in connection therewith is to determine whether rates made as outlined, or on other principles, are or are not confiscatory, and the fact that in one of the leading rate cases of recent times, where valuation has had its day in Court, the conclusion was finally reached that one of the three roads under consideration would be justified by its valuation in charging higher rates than the other two, if it could obtain them, which, of course, as a practical matter, it cannot.

With this in mind, it seems as if the Society as a body would be exceedingly ill-advised to urge as the result of its deliberations a conclusion that railroad rates are or can be based on valuation, and the report, therefore, should be confined to public service properties (on consideration of which it is mainly based), which by their very nature are limited and protected in their field of activity, and no attempt should be made to cover rate-making for railroads which is based on far different considerations.

WILLIAM W. CREHORE,* M. AM. SOC. C. E.—Broadly speaking, the subject adopted by the Committee for its report, namely, Valuation for the Purpose of Rate-Making, the subjects of certain papers which have been presented to the Society recently, namely, The Appraisal of Public Service Properties, The Physical Valuation of Railroads, The Depreciation of Public Utility Properties, and similar topics, all involve the one important question: How shall a fair basis be determined for the rates to be charged by public utility corporations? Mr. Crehore.

The Real Question.—First, what is the true basis for reasonable rates, and, second, on that basis, what are reasonable rates? When these two questions have been answered satisfactorily, the object of the present discussion will have been attained. The different phases of this subject are introduced to lead by different routes to different conclusions, but these conclusions must furnish an answer of some kind to the question: What are reasonable rates?

The point has been raised in this discussion that a definition of the term "value" must be adopted before it can be used intelligently, that the Courts have defined it in various ways, and that many of those taking part in this discussion have defined it at variance with the Courts. The speaker strongly recommends that some standard

* Previous discussion by Mr. Crehore appeared in *Proceedings* for February, 1914, on p. 380.

Mr. Crehore. definition be fixed for the term, "valuation", which will be the understood meaning of the term, applicable to all purposes for which a valuation is to be used. To this it will be objected that years of custom and Court decisions have established a precedent requiring property to be valued for taxation in a different way from that required for rate-making or for acquisition. So far as we are bound, by law and Court decisions, to follow certain courses of action, there can be no discussion as to our duty, but being thus bound should not prevent our careful consideration of the propriety of existing laws and customs, or an attempt to change such laws and customs whenever it can be shown that a change would be beneficial. To set aside this whole question with the statement that the conclusions of one's opponents do not conform with existing laws and customs is not argument. The expediency of asking for a change of prevailing law involves a discussion of the effects which would follow such a change, and it is certain that no change for the better will ever be adopted unless convincing arguments are brought forward to uphold its expected advantages, and necessarily such a discussion must precede the campaign of education required to bring about the change.

The Term "Physical Value".—To approach this question with an open mind it should not be taken for granted that a proper basis for rates need be valuation of any sort whatever. The real valuation may be found to be something which is not at all suitable for a rate basis. Entirely apart from the question of valuation, and after its determination, it should then be decided whether or not the true valuation is a proper basis for rates.

It seems to have been pretty well agreed by the majority in this discussion that the intrinsic or tangible value of any property is not its "Sales Value," nor its "Original-Cost-to-Date Value", nor its "Cost-to-Reproduce-New Value", but that it is its "Cost-to-Reproduce-Less-Depreciation Value". The term "intrinsic value" denotes the inherent value, the value residing in the property itself, the tangible value, as distinguished from extrinsic or intangible values, such as good will and other elements of value attaching to the property by virtue of its association with other property, or for some other reason, but not values inherent in the property itself. It will be noted in passing that, under this definition, depreciation could not logically be included in the intrinsic or tangible value of property.

The preceding general discussion has been characterized by much indefiniteness on the point of what constitutes the physical value. Some have attempted to make the term cover intangible values besides the intrinsic value, and others have restricted its use to the intrinsic value alone. Still others have included within its scope all the intrinsic value and a part only of the extrinsic values, expressly excluding certain intangible values. For the most part, it seems to be the general intent

to restrict the term "physical value" to the intrinsic or the actual tangible value of the property. Mr. Crehore.

The sales value usually includes something for extrinsic or intangible values, and yet, depending on the prosperity of the business or the value of the property due to its association with or separation from other properties, the sales value may either exceed or fall short of the intrinsic value of the property. In the majority of cases, the selling value constitutes a contract agreement between the two parties interested, and their individual circumstances at the time of the sale will sway the price one way or the other in spite of any estimate that can be put on such property. The selling price, however, is influenced most of all by the earning capacity of the property; and, as has been pointed out several times, as we are seeking a basis for regulating the earning capacity, it would be reasoning around in a circle to offer to determine that basis by the very element we wish to make dependent on it.

The merchandise in every modern store is inventoried by the Cost-to-Reproduce-Less-Depreciation Method once a year or oftener, and yet no one would think of buying out a prosperous merchant simply by paying him for the physical value of the goods on his shelves. There must be a further payment for certain extrinsic values, including the good will of the business. The feature of good will and other extrinsic values, in the case of a highly prosperous monopoly, may constitute by far the largest asset, the intrinsic or physical value of its real property being immaterial by comparison. What is it, then, that gives any extrinsic or intangible value to property? It is the earning capacity tempered by the risk involved in its ownership. When a man buys stock, all he wants to know is how much income he can expect from it, and the reputation of the corporation. Satisfactory answers to these questions do not need to be supplemented by information as to the amount of capitalization, the value of the assets, etc., etc. The corporation may not have any assets in sight except the good will of the business, and yet the stock may be a first-class investment.

Physical or Intrinsic Value Inadequate as a Basis for Rates.—It is plain that the physical value, that is, the intrinsic value of property of all kinds, can be estimated with a fair degree of accuracy, but that the extrinsic or intangible value depends for its size directly on the earning capacity of the unified business taken as a whole. As we cannot seek a basis for regulating the earning capacity in something that is dependent on that earning capacity for its very existence, there is no part of the value available as a basis for rates except the bare physical or intrinsic value of the property. As already stated, it is not necessary that valuation of any sort be made the basis for rates, but it is apparent that if rates are based on the bare physical valuation of the property, without any allowance for return on certain intangible values, the return on the investment will not be adequate.

Mr. Crehore.

Many items of intangible value have cost the investor large sums of money. Articles of trade often have to be given away to introduce them to prospective customers. Advertising is very costly, and some business enterprises require two or three years of advertising before they can show a profit on their sales account. There is nothing to show for these and other large initial expenditures except intangible values, generally designated as the good will of the business. Yet, in some lines of business, the good will is the chief asset, because the actual amount of tangible property required to keep them going is so very small. The professional man with a large and lucrative practice would certainly not care to sell out for the value of his tools or other physical assets required to keep up his business.

A toll bridge company may find at some stage of its existence that the physical value of its bridge and other property less depreciation is $20 000, and yet may be able to show annual profits which, if capitalized at their present worth and treated as an annuity, would require a fund of $30 000 from which the equivalent of the present actual profits of the company could be withdrawn annually so that the fund would just be exhausted at the expiration of the life of the company's charter, the probable life of the bridge and the company's charter being coterminous. Such a fund would then be the proper measure of the total value of the property, including the intangible as well as the tangible.

Although the examples just cited may not be strictly applicable to the case of a public utility, it is well known that there are many hundreds of items which enter into the cost of launching any large enterprise, whether it be public or private, and some of these items never produce any beneficial effect on the business, while others more than make up for them and give it a very material intangible value, which is just as real and costly as though it were possible to see, feel, and estimate its items.

It is apparent that the earning capacity of any property is the vital point in the determination of its total value, both extrinsic and intrinsic; yet the intrinsic value only can be determined by the Cost-to-Reproduce-Less-Depreciation Method. As intangible values are indeterminate, except by means of the earning capacity, which is what we are seeking to fix, it therefore seems reasonable to conclude that a just and equitable value on which to base rates cannot be found. It must be assigned, or approximated, or guessed at. In truth, the problem of evaluating public utilities is the reverse of the ordinary problem met with in dealing with private corporations, and begins with a determination of the minimum return that can be offered to attract capital. A private corporation, on the other hand, is in business to make all the money it can for its stockholders, and is supposed to be regulated by the power of competition.

The Fair Allowance as a Basis for Rates in Excess of the Physical Value.—Mr. Grunsky, in his paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates", said: Mr. Crehore.

"Intangible values, of whatsoever nature, result from high earnings. In the case of public service corporations, they are arbitrarily created by agreeing to, and permitting, rates which produce a revenue in excess of the ordinary return on safe investments. They do not exist unless the rates are higher than those which would produce net earnings equalling an ordinary interest return on the properly invested capital."

An "ordinary interest return on the properly invested capital" is elsewhere spoken of by the same author as "the return on ordinary safe investments" like savings bank deposits or other investments of like character. Mr. Grunsky, in course of his able presentation of the subject, takes the ground that intangible values should be disregarded in making appraisals for rate-fixing purposes, but that the net earnings of a public service property should in some measure exceed the return from ordinary safe investments. They should, indeed. It is certain that capital could not be attracted at all to our public utilities unless there was a prospect of greater return than is offered by savings banks or Government bonds. As just stated, the public utility problem begins with a determination of the minimum return that can be offered to attract capital. In explanation of his position in this matter, Mr. Grunsky says:

"reference may again be had to the case of an owner of a public service property who invests only borrowed money. If he receives only such interest on the investment as he must pay to the bank, he will have rendered a service without compensation, except such as he may be allowed in salaries, under operating expenses. In such case, it would be a proper business arrangement to compensate him for the risk of loss which he assumes, and for his management, and to make this compensation in some measure proportional to the net earnings. If the owner in the cited case is a stock company, this compensation will be the only element giving value to the capital stock of the company".

As one method of fixing a basis for rates, when finally the physical or tangible value itself has been found, it will be possible to make rates on it so large that the return to the investor will include something for certain assumed intangible values, that is, the percentage of return may be made any desirable degree larger than the return on ordinary safe investments. Perhaps this is the method which would be adopted by those who contend that the physical valuation alone should be made the basis for rates. The trouble with this is that to allot a fixed percentage of the physical value as the return on the intangible values is to assume that these two values always bear the same ratio to each other, which is very far from true. Certain it is that, without some

Mr. Crehore. allowance in the rates to remunerate intangible values, there would be little attraction for investors. The necessity for adding something to the purely physical valuation when used as a basis for rates is apparent; but, as to what that something should be, there has hitherto been no agreement. Although the different classes of public utilities would require different treatment in this regard, the speaker believes that some average basis could be worked out for each class by itself that would provide an adequate remuneration for those intangible values which originally caused the investors some material outlay.

The Investment as a Basis for Rate-Making.—One of the methods for fixing a basis for rates, seemingly overlooked in this discussion, is to determine the investment. This is not at all what is meant by the Original-Cost-to-Date Method of valuation. There is a clear distinction between the actual cost of all the property purchased by the corporation and the actual amount of cash or property contributed to its treasury by the investors. If the latter could be ascertained by any practical method of accounting, it would be the ideal basis for rates. There would be substantial justice by this method, for the application of it would work both ways. It would take care of the investors' rights, in the event of mismanagement or unwise expenditures of the corporation's funds, and it would prevent burdening the rate-payers with a load of fictitious capitalization. It would also return to the rate-payers the benefit to be derived from a reinvestment of any portion of the earnings and the unearned increment on land values, removing these valuable sources of appreciation from the investors and giving them to the community, where they rightfully belong.

Choice of Valuation by the Owner.—In view of the many unsatisfactory features surrounding and pertaining to all methods of valuation for rate-making hitherto considered, and in view of the enormous amount of labor and expense required to put any one of them into execution, the speaker again suggests for serious consideration the expediency of taking the owner's valuation of his own property and using that as a basis for rates. There should be an end to all juggling with the term "valuation". There should not be one kind of valuation for rate-making, another for acquisition, and another for taxation purposes. If the owner is made to look at the question from both sides, he is far and away the best expert as to the true value of his own property. The way to make him look at it from both sides is to require him to pay taxes on the same valuation that he is allowed as a basis for his income. Within certain limits, it should be the province of public authority to prevent grossly irregular and inappropriate valuations, but, in general, the owners of public utilities should be required to name their own valuation, with the understanding that it was to be used as a basis for rates as well as for taxation, or for acquisition, or for any other purpose.

Think of the problems it would eliminate. Who ever heard of a public utility paying taxes on a fictitious valuation of any sort if the opportunity had been given to swear them off? Would any public utility care to pay taxes continuously on the proportion of intangible values which it now claims should be rewarded in the rates? If valuations continued to increase, due to the capitalization of earnings, either past or future, at least the rate-payer would have the satisfaction of taking his share of the earnings on these increases through the tax budget; whereas, if utilities in general neglected or refused to declare these additions to their property for taxation, the rate-payers would stand exactly in the same position as if they owned these increases outright, and would enjoy a corresponding reduction in the rates on that account. In the last analysis, the public would remain in control, both of the tax budget and of the rate-making, so that any consensus of action, by the utilities overbalancing the situation either one way or the other, could be readily corrected at the proper time. Mr. Crehore.

At least, while we are waiting the many years it will take to get a physical valuation of our public utilities, let us try the alternative of letting the owners set their own valuation, with the common understanding that it is to be used for all purposes where required.

There is another point to be considered: After the proposed stupendous task of valuation is finished and the returns are in, how long will it be before conditions will have changed so much as to require another similar valuation? General conditions will certainly call for a readjustment of rates periodically, and how can this be done intelligently on the basis of an out-of-date physical valuation? Is this task, which is already recognized as Herculean, going to be repeated as often as a readjustment of rates is required? Rather let us have the task divided up and discharged by the owners themselves who are intimately familiar with the conditions, and then it can be repeated as often as the convenience of the public and the interests of the utilities demand.

Frequent Rate Regulation Undesirable.—The magnitude of the interests involved renders it extremely undesirable that rates should be changed too frequently. To readjust them once in 5 years, immediately after the utilities had filed their valuations, would probably be often enough. It might be too often. It would be preferable to have the period between adjustments too short than too long, however, because, if not long enough, the opportunity may always be availed of to continue the same rates for another period. Reasonably frequent periods would give the utilities a better opportunity to readjust their valuations with regard to all the elements to be considered.

Excessive Bonding and Capitalization of Earnings.—Recently, the Interstate Commerce Commission called attention to the manipulation by the Chicago, Milwaukee and St. Paul Railroad Company of the accounts of the Puget Sound Railway Company, a subsidiary of the St.

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Paul, saying that it was done for the purpose of misleading the public and creating a market for its new bonds. The Commission claimed that this was unnecessary, as the Puget Sound Company could show reasonable profit on its operations without any manipulation of the books, and was in fact on a sound basis, both financially and physically. The President of the Puget Sound Company forthwith explained to the public that the only manipulation of which the Company might possibly be accused was rendered necessary by an effort to obey the laws of the State of Washington, and, in so doing, the bookkeeping was forced to be a little out of compliance with the mandate of the Interstate Commerce Commission. A significant fact of the President's statement was that the Puget Sound Company's original capitalization had been \$3 000 000, but when the construction of the Puget Sound Extension was nearly completed a mortgage for \$200 000 000 was found to be necessary, and such a mortgage was thereupon issued. The excuse for the irregular bookkeeping previously mentioned was that, under the laws of the State of Washington, it was impossible to issue a mortgage for more than twice the par value of the capital stock of the Company, and, therefore, the capital stock had to be jumped suddenly from \$3 000 000 to \$100 000 000. Naturally, it would require an agile management to stretch out the hitherto reasonable and sufficient return on \$3 000 000 so that it would look anything like adequate for a return on \$100 000 000 in addition to the carrying charges and amortization of a \$200 000 000 bond issue.

Now, the phase of this situation which challenges our attention is that any company or individual should have the hardihood to ask for a loan of \$200 000 000 on property capitalized at only \$3 000 000. The first reply to this is that the property was worth much more than the amount at which it was originally capitalized. Let us assume that it was. Let us assume that, by the reinvestment of earnings since the time of its organization, the property had attained an actual value of \$100 000 000, the figure at which it was finally capitalized. Even then, the amount of the proposed mortgage was twice the value of the property, and its issue was specially sanctioned by a law of the State. Is there any private business, corporate or other, the managers of which would ask for and expect to get a loan amounting to twice the amount of the security offered as collateral? Rather is it more reasonable to expect that even the best terms obtainable on a mortgage of this kind would be one-half or 60% of the value of the property, instead of twice its value. Yet there are those who would have us believe that a public utility is on no different footing from that of any private corporation.

It seems that the financial managers of public utilities always rely confidently on a steadily increasing valuation of their property through the reinvestment of earnings. They assume that out of future earn-

ings they will always be able, not only to cover the current depreciation, but also to carry and amortize bond issues far in excess of the present value of the property, and pay a profit besides. In other words, they use bond issues instead of subscriptions to the capital stock, because in so doing they are getting capital free of charge from the public, and whatever profit they can make over and above the interest and sinking-fund payment on the bonds is so much clear gain without the expenditure of a penny to obtain it. Generally, in years gone by, the steady increase of earnings justified this tendency to rely on the future; but in recent years the rate-payer has interposed his veto against supplying capital as well as income. When the appraiser comes along to ascertain the physical value of such property and finds large accessions to it due to the investment of the proceeds of bond issues, the question naturally arises: Should the valuation of this property be credited to the corporation and be made the basis of rates charged to the public in addition to the carrying charges and amortization of the bond issues? The plain statement of the question indicates the injustice of the process, and yet this was, until very recently, and is even now, when not prohibited by law, one of the ordinary methods of financing a railroad. Mr. Crehore.

The reinvestment of earnings over and over again for many years and the discounting of future earnings by bond issues have increased the value of railroad properties at the expense of the rate-payers all over the country far beyond the stockholder's actual investment, and often beyond the capitalization. The discussion of rate allowances must take into consideration the fact that an enormous percentage of the values now found to exist in public utility properties was contributed by the public in the form of excessive rates, and, instead of using this part of the valuation as a basis for increased rates, it should in justice be used as a basis for the reduction of present rates, so that the actual investors in the property would be adequately remunerated, whether they are the rate-payers or the subscribers to the capital stock.

There are those who say, in reference to this matter: "Oh, let bygones be bygones, but let us legislate for the future so that the public may not be forced to contribute capital as well as income to the owners of our public utilities". In urging this view of the matter, it is argued that the present stockholders would be the actual sufferers, as they have paid the current prices for their holdings, and the current prices were based on the earning power of the properties averaged over a period of years. It is this fact that has halted the Interstate Commerce Commission in its treatment of the question of capitalization of earnings, and it is this phase of the matter which perplexes all who have given attention to an analysis of this subject. To base rates on the present valuation of these properties would be a fair solution of the problem for the present security holders, but would be penalizing the

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rate-payers by compelling them to contribute income to the security holders for that portion of the value of these properties for which the rate-payers themselves furnished the capital. For years the reinvestment of earnings, that is, of capital supplied by the rate-payers, has provided our public utilities with an argument for increasing the rates, whereas, on the other hand, it should be an equally potent argument for reducing the rates, if the public was honestly given the benefit of its investment.

The public's plaint, however, does not end here. It has been shown that often and often bond issues by public utilities have been refunded and carried indefinitely long after the property originally provided by their proceeds is worn out or has become obsolete. No effort has been made to retire these loans with the extinction of the property representing them, but they are refunded again and again, and are claiming their share of the earnings of the corporation for something which has long ceased to exist. If the depreciated or obsolete property has been replaced out of the earnings without an accompanying retirement of the bonds, is it not plain that the property and the bonds cannot both claim their share of the earnings without putting a double load on the rate-payer? Is it not just as clear, also, that the process of rolling up the value of the property year after year, like a big snow-ball, out of the earnings of the corporation, calls for continually increasing rates to provide additional income for the security holder derived from the rate-payer's own investment? Is it not evident that the more the rate-payer helps the corporation along on its capital account, so much the more is he penalized by having to pay rates which will cover an adequate income on the ever-increasing valuation? After some years of this cumulative process, the public is now paying rates which are enormously in excess of a reasonable return on the capitalist's original investment; yet the property value is there, and would appear by whatever means the valuation was undertaken. Furthermore, if this process continues in the future, there is no end to the advancement of rates; the more they advance the more they will have to advance to cover the income on their own advancement. It is automatic.

If the rate-payers are also the owners, then the capitalization of earnings is a perfectly proper procedure, and has the effect of automatically reducing the rates any way; but, if the rate-payers are not the owners, it certainly will not be claimed by any one that they should be called on to furnish all the capital used in the business. Then just how much of this capital is it proper and right that the rate-payers should supply, and how much should be furnished by the stockholders? The answer to this question will be one element of the basis for fixing the rates.

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Water.—So far as the rate-payer is concerned, however, this load of furnishing the capital for public utilities is negligible in comparison with the one he is carrying as a result of fictitious valuation. This feature is one that the present campaign of rate-fixing is designed largely to correct, and the present holder of the securities will have to stand the "squeeze." There is no help for him in this regard, nor should there be. He may have been innocently drawn into the investment without a proper investigation, or he may have relied on the movement of stock quotations in forming his judgment of the value of his intended investment, but in any case he has lost by investing in something of which the value was fictitious, and there is no more reason for protecting him in this matter than for protecting the investor in a private banking business who found that foreclosure on certain loans hitherto believed to be impregnable had developed a large shrinkage. To attempt to save the present stockholder from loss in these cases would be saddling forever on the public an almost intolerable burden of fictitious values.

Considering this question broadly, however, such cases of overvaluation as have resulted from the rascality or deliberate manipulation with intent to deceive on the part of one man or a small coterie of men, might receive special treatment in order that the security holders in these enterprises may not be entirely robbed. It might be found feasible in some of these cases to divert some of the surplus earnings now being reinvested as capital and apply them to a gradual reduction of the holdings of innocent stockholders who were the victims of the rascality of the managers. This, however, comes more properly under the head of financial management, and is outside the topic now under discussion.

Land Values as Excess Earnings.—Even if the percentage of excess earnings taken from the public to be used as capital by the public utilities is small, it makes a large difference to the capitalist in the amount of his actual investment. Constantly adding to his holdings from the capitalization of earnings, the original investor finds his income increasing in times of general prosperity with the acceleration of a compound interest account, without effort or expenditure of his own. References have been made by several of those participating in this discussion to the treatment of land values in connection with public utilities, and Mr. Sparrow has handled this phase of the subject ably and clearly. Why should not the appreciation of land values be treated like values resulting from a capitalization of earnings? The unearned increment in the value of land is no more and no less a gift from the community than is the increase of capital from excess earnings. The added value of public utility property due to reinvestment of a part of the earnings each year has precisely the same effect as the appreciation of its land values, adding to the assets of the investor without

Mr. Crehore. any expenditure or effort on his part. The only distinction that can be made between the two is that the unearned increment of the land costs the community nothing, whereas all additions to the capital account out of the earnings come directly from the community in the form of high rates. The speaker fails to see any reason why both these questions should not be accorded the same treatment.

It is just as true of any private corporation as it is of a public utility that the reinvestment of a part of the earnings as capital is a free gift from the community, but a private corporation maintains its rates (or is supposed to maintain them), in open competition with all others in the business, whereas a public utility is under contract with the State to perform certain functions in return for which it is accorded a praccal monopoly in the district which it serves.

Is the Capitalization of Earnings Confiscation?—Opponents of the proposition to deduct the accrued depreciation from the value of public utility property in establishing the rate-fixing value are very sure that to exclude this depreciation from the rate basis would result in a form of confiscation of the owner's property. This has been loudly proclaimed, and argued with some force. Do those who take this position also find any evidence of confiscation of the rate-payer's property in the capitalization of earnings? If it is taking property from the corporation without due process of law, to exclude from participation in the earnings such capital as has been used up or has become obsolete, or such portions of existing capital as have been allowed to deteriorate and have not been kept up to their full valuation or are without the means to supply this lack, then it is no less confiscation to take from the rate-payer funds to be used as fresh capital on which no part of the return is shared with the rate-payer. The speaker does not think the term "confiscation" applicable to either of these processes, but believes that they are on the same footing with respect to the import of that term.

Efficiency Its Own Reward.—As intimated by the Committee, one of the most important of the intangible values, and one having an amount totally disproportionate to the physical value, even among utilities of the same class, is the item of efficiency. Of two public utilities having practically the same physical valuation, one may be far more efficient that the other in handling its business. A uniform percentage based on the physical value in each case would remunerate each of them alike, taking no account of the superiority of one over the other. It appears to some that such treatment would penalize the more efficient company and put a premium on inefficiency; but, would not the more efficient company find its reward in the resulting lower cost of operation, and would it not, even at the same rate of remuneration as its less efficient rival, by reason of its very efficiency, have a greater margin of profit left for its owners?

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A uniform percentage on the total physical valuation, as the basis for uniform rates, would include an allowance for operating and administrative cost, so that the uniform rates would mean greater profits to the company having the lower cost. Of two utilities covering practically the same territory and serving populations of equivalent size, one might have its capital invested in thoroughly modern equipment of higher value and greater efficiency than that of the other. Appraisals of the two would show for one a larger physical valuation than for the other, and, consequently, the total general valuation for establishing the rates would be higher the more of these modern plants there were. If, as before, the allowance for rates was a fixed percentage which covered the total cost of operating as well as the reward to capital, then the more modern plant, with an equipment having admittedly a higher efficiency than that of the older plant, will reap a further reward for its investors by the lower cost of its operation. In the same way that all other lines of business are compelled by competition at certain times of their existence to discard old and worn out or half worn apparatus for new and modern appliances, just so a public utility finds its own reward in making the change.

It has been suggested in this discussion that if rates are adjusted for the highly efficient competitors in the same territory, the poorer of these will lose until they meet the competition. It is understood, of course, that rates will be based on an average of conditions and valuations representing all utilities in the same territory. On such an assumption the writer agrees with Mr. James that the stronger and richer competitors might be required to pay the State, in the form of taxes, all the excess over the earnings of their weaker rivals, such rebate to the State being arranged on some sliding basis that would preserve the required incentive to efficiency intact; and it would be well to go farther, and limit the time within which the poorer and weaker would be required to put their equipment on a modern efficiency basis or else have the protection thus afforded withdrawn. Rates must be made uniform, whatever system is adopted, and cannot be based on conditions which would suit a single utility or group of utilities in any one class.

Other Problems Concerning Rates.—There are many other problems more important in their results to the public than the problem of finding a rate basis. After the basis is found, rates must be adjusted to it; but, in the process of adjusting them, there are thousands of classifications of which to take account. In the case of railroads, the most complex problem of the whole is that of making the various classifications. These classifications now furnish the basis of nearly all the discrimination prevalent throughout the United States, and it is these classifications and other discriminations, rather than the economic law of supply and demand, that determine in a large number of

Mr. Crehore. instances the movement of traffic throughout the country. It is plain that freight rates cannot be made altogether proportionate to weight or bulk. Under such a uniform rule, many very cheap products could not be shipped at all, because it would cost more to transport them than they are worth. Even now, where such shipments take place, and the railroad company does not collect in advance, it may never get its money at all, because, on refusal of the consignee to accept the goods, the railroad would have only the value of the shipment to reimburse it. There must, therefore, be enough more profit in handling the better and higher class of goods to make up for the lack of profit in handling the cheaper goods.

This classification problem is a very complicated one, and this fact has been taken advantage of by the railroads very extensively to assess the public for all the traffic would bear, regardless of uniformity. The Interstate Commerce Commission has done a large amount of work in the past in adjusting these classifications, but still more remains to be done, and there always will be more to do. New questions are continually arising which cannot be foreseen, and, with the railroad companies presenting only one side of the case, it is the duty of the public to present its own side as clearly as possible for the benefit of those in authority who have the final adjudication of these matters.

Mr. Maltbie. M. R. Maltbie, Esq.*—There is one thing in the report which has not been mentioned but which seems to be of special value. The Committee has pointed out that the question of the amount upon which a company is entitled to a fair return, whether cost-to-reproduce-new or depreciated value, is inseparably connected with the method of computing the allowance for annual depreciation. Many writers have failed to appreciate the relationship between these two, and have used one method for computing accrued depreciation and another for computing the annual depreciation. The additional discussion by Mr. Stearns has emphasized the point made in the original report, and demonstrates conclusively that one cannot fairly adopt one method for determining accrued depreciation and another for determining annual depreciation. It would be grossly unfair, as shown by Mr. Stearns, to adopt cost-to-reproduce-new as a basis for reaching fair value, and the straight-line method of depreciation in computing the annual allowance.

There are many other factors which the Committee has similarly treated, and indeed, it is probably doubtful whether there are many factors (perhaps there may be a few) where it is possible to settle one question or determine one factor without reference to the determination of others. If the Committee had emphasized and reiterated no other principle, it would deserve our thanks and commendation, for writers and experts have been prone to slip from one thing to another without appreciating the interdependence of the different factors.

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Mr. Maltbie.

The fundamental question in rate cases is the rate, and the determination of a fair and reasonable amount collected from the consumers or those using the utility. In order to arrive at this, it has been customary to determine the fair value of the property and the rate of return; but, after all, there is no one yardstick which will apply to all cases, and it is only by threshing out the various questions, by clearing the atmosphere, which is often befogged, that it is possible to determine the final outcome; and two plans may bring the same result, but one may differ from the other in practically every point. The report of the Committee indicates that this idea was fully appreciated, and it has attempted with great care and forethought to elaborate a statement of principles which will do justice to the public and to the company.

Mr. Floy.

Henry Floy, M. Am. Soc. C. E.—Before taking up the discussion of the Committee's report, it might be well to call attention to the impracticability of the method of valuing public utility property proposed by Mr. Crehore. This is to let the owner place a value on his property, with the understanding that the rate of return allowed and the taxes to be paid thereon shall each be figured at the same percentage of the value of the property. Although not having any investments in public utility properties, the speaker thinks that all the utilities would be glad to accept any such basis of valuation and pay taxes to the extent of 10, 20, or even a higher percentage thereon. A little reflection will show that, as taxes are paid by the public—the utility merely acting as a collector—the corporations would be perfectly satisfied to pay any amount in taxes, provided the same amount was allowed them for dividends. It will be seen, therefore, that such a method of valuation would be not only impracticable but manifestly unfair to the consumer.

With regard to the suggestion of Mr. Lavis that the membership of the Committee be increased by the addition of three railroad men, the speaker would heartily endorse such procedure, stipulating, however, that the new members be specialists in the matter of appraisals and rate-making—not merely construction or operating engineers. A further suggestion would be that the Committee have added to its membership at least one capable attorney who has made a special study of the decisions of Courts and commissions in relation to the matters covered by the report of the Committee.

An examination of the Committee's report shows the painstaking effort and high character of the work of its members. Credit therefor, and the thanks of the Society, are due the Committee, even though we may not agree with its conclusions.

The title of this report would lead one to expect that its recommendations could be followed in appraising existing utility properties. As a matter of fact, the report deals largely with methods proposed to

Mr. Floy. be applied to corporations which have been very recently created or which may be established in the future.

The serious question before engineers at present, is, not what possible theories and methods may be developed for ascertaining the value and determining the depreciation of utilities yet to be created, but rather what methods of valuation shall be followed in appraising the many existing and long-established utilities.

Probably many engineers—and certainly the speaker—would not differ with most of the Committee's report and recommendations, provided such recommendations are applied only to newly created corporations in which investors have placed their money after due notice as to the methods which would be applied and the returns which would be securable therefrom. Few will find fault if a commission, following the Committee's recommendations as to the "tango" treatment of return—a dip at each successive step—can induce capital to invest under such proposed rulings, but it is absolutely unfair and illegal to attempt to saddle untried, theoretical methods on innocent owners, who have invested their money under conditions and upon assumptions entirely different from those now proposed by the Committee.

On this account, much of the report will be of but little help to the Profession in determining proper methods of valuing long-established, existing utilities. To be sure, the report recognizes the necessity of using the so-called "reproduction-cost-new" method of getting at the value of long-existing utilities, but the practical application of such method is only generally indicated.

Passing over the many excellent points in the report that relate to making up an inventory, preliminary, engineering, and development expenses, etc., it would seem as if the Committee was attempting, without sufficient reason, to sweep away many of the fundamentals that have been generally recognized and are now accepted.

The Committee seems to assume that interpretation of the decision of the Supreme Court in the Knoxville Water Company case is the only true gospel. The Committee then proceeds to argue that other leading decisions (more numerous, and as to which the interpretation is less debatable) of the same Court are in error and must be corrected to conform with the Committee's views as to the proper method of making valuations.

As is quite generally known, the Knoxville Water Company case was not adequately prepared, or well presented, in comparison with present-day knowledge of these matters, consequently, the resulting decision and many of its dicta have been erroneously accepted, both from misapprehension of the facts, and misinterpretation of the Court's meaning. If the Committee hopes to attempt the reversal of any Supreme Court decisions, it should attack some of the principles

claimed in the Knoxville decision, rather than those others, which relate to cases that were more adequately prepared, better presented, and more recently decided. Mr. Floy.

In considering any decision of the Supreme Court, it must be borne in mind that the Court is passing on the particular case in question, and any conclusions reached are based on the peculiar circumstances surrounding that particular case. Consequently, too much importance should not be attached to the decision of the Supreme Court in some particular case when attempting to arrive at fundamental principles, which may be applied generally. Although it is true that the last word in any particular instance rests with the Supreme Court, and that such Court is given to following precedent, nevertheless, it cannot be presumed that the Court will seek to perpetuate what may be demonstrated to be a wrong, merely to avoid reversing itself.

No question can be regarded as finally settled, until settled equitably. The reference by Mr. Humphreys to the decision of the Supreme Court, in the Consolidated Gas case, that to maintain a pressure of 2½ in. of water in the gas pipes in a distributing system, "the mains and other pipes would have to be strengthened", is so contrary to fact that eventually the Supreme Court must reverse itself in this matter. In the same way, the statement of Judge Hughes, in the Minnesota rate cases, to the effect that "We also think it was an error to add to the amount taken, as the present value of the lands, the further sums calculated on that value, which are embraced in the items of 'engineering, superintendence, legal expenses', 'contingencies' and 'interest during construction' "; or again, the statement in the same decision, "It is impossible to assume, in making a judicial finding of what it would cost to acquire the property, that the Company would be compelled to pay more than its fair market value. It is equipped with the governmental power of eminent domain", is so contrary to common knowledge and experience in railway construction that the establishment of such principles cannot be accepted, even when attempted by the Supreme Court.

The Committee seems to require some legal support for making deductions from reproduction-cost-new, and, therefore, ties to the Knoxville case, and has developed a unique theory of estimating and providing for depreciation, the allowance therefor being stated to be "in effect, a payment from the rate-payer to the corporation of a part of its investment" (page 32). This is certainly a novel definition, and one that is contrary to accepted practice. The Committee, without apparent warrant, remarks (page 72), with reference to its proposed "Equal-Annual-Payment Method of Determining Depreciation Allowances", that "this method conforms to the decisions of the highest Courts". No references are given as to what Court decisions are

Mr. Floy. considered by the Committee, to justify their proposed method of annually increasing payments to be made by the rate-payer on account of depreciation and decreasing his payments for fair return. The Supreme Court certainly disapproves any such method as that proposed by the Committee, and the speaker suggests that the Committee produce the opinion of a competent attorney to the effect that any Court decision rendered to date is based on any such theory of ascertaining present value as proposed by the Committee's "Equal-Annual-Payment Method".

It is conceded that the decisions of some Courts hold, that, in determining the value of property on which to fix rates of return, deduction from cost of reproduction new, on the basis of a more or less theoretical computation, should be taken into account, but this is not the ruling of the United States Supreme Court. This particular question, with reference to whether the amount of depreciation of physical property was to be determined from actual inspection or from calculations based on life tables, was squarely involved in the Consolidated Gas case and definitely passed on by the Master appointed to take the testimony in that celebrated case. The total valuation of that property was some $56 000 000. It was shown that an expenditure of $604 988 for "repairs" (less than 1%), would make the plant as good as new, and this is the only sum that was deducted from the reproduction cost new, notwithstanding it was most vigorously contended by the plaintiff in that case that a further deduction of millions of dollars should be made for theoretical or accruing depreciation. The Master, in answer to this contention, says:*

"For the purpose of determining present value, however, particularly on the basis of cost of reproduction, the method followed by Mr. Marks [witness for the plaintiff who used theoretical depreciation based on life tables, similar to those proposed by the Committee] does not commend itself. It appears from the record, without substantial dispute, that while certain of the plants and apparatus may not be in perfect repair, they are, as a whole, in efficient operating condition, and that a large proportion of their capacity is represented by the latest pattern of water gas apparatus installed within the last few years."

The Master, in this case, in dealing with the alleged necessity for a reserve fund to provide for "final renewals" when the life of the apparatus should expire, says:

"Of course, the requirement of such 'final renewal' provision affects in no way the present value or efficiency of the plants of the company as operating concerns, except to the extent of the repairs [$604 988 above mentioned], which would be required to make the operating plant as good as new."

* Master's report, p. 110.

Mr. Floy.

The Circuit Court and the Supreme Court sustained in all respects the valuation fixed by the Master in this case, except on the question of franchises.

It is thus seen from the foregoing decision that property which is in good order and operating efficiently, although not new, need not necessarily be depreciated, at least in rate cases.

It is sometimes contended that the decision in Knoxville *v.* Knoxville Water Co., 212 U. S., 1, decided the same day as the Consolidated Gas case, holds a different doctrine, but a careful reading of that case, in connection with the Master's decision, shows otherwise.

Depreciation which can be determined by "a detailed examination of the property as it stands to-day,"* covers that deterioration due to inadequacy, obsolescence, deferred maintenance, or property not actually used in the service being rendered. It is these various classes of depreciation just specified, and not theoretical calculations based on office records as to date of installation, that are referred to by the Supreme Court in the Knoxville and other cases where the specific statement has been made to the effect that depreciation must be deducted in determining the values of property to be used in fixing rates. In the Knoxville case, the Master stated that he had added "complete and incomplete depreciation" to the present value of the surviving parts, in order to obtain the total value used as his basis for fixing rates. Consequently, the Supreme Court very properly criticized such procedure. It was equivalent to taking the depreciated value of the existing property and adding thereto the cost of all property which has been used in the past and which no longer existed, the value of which under adequate rates, should have been repaid to the investors out of revenue as a part of operating expenses, during the life of the now superseded property.

The Supreme Court says that the Master erred in adding the sum of "complete" and "incomplete" depreciation to the depreciated present value; it does not state that he would have erred in adding only "incomplete" depreciation to the depreciated value. The Court could not have considered that adding the amount of "incomplete" depreciation to the present depreciated value would have been error, otherwise it could not logically have stated that renewals must be paid for out of revenue as part of operating expense, for,

"if a different course were pursued the only method of providing for the replacement of property which has ceased to be useful would be the investment of new capital and the issue of new bonds or stock. This course would lead to a constantly increasing variancy between present value and bond and stock capitalization, a tendency which would inevitably lead to disaster to the stockholder or to the public, or both."

* Master's report, Knoxville case.

Mr. Floy. It is evidently the thought of the Supreme Court that present value and the amount of capitalization, as represented by stocks and bonds—that is, the investment value—should correspond, or tend to equal one another, which would not and could not be the case if "present value" were taken to mean present theoretical depreciated value. This view of the Supreme Court is in harmony with its decision in the celebrated Consolidated Gas case, where only observed, actual, evidenced depreciation, obsolescence, or deferred maintenance was deducted.

Nor does the decision of the Supreme Court in the Minnesota rate cases decide that theoretical depreciation should be deducted from reproduction new to arrive at present value as a basis for rates. This decision, so far as it dealt with the questions of valuation, has been quite generally criticized by men best equipped to form an intelligent judgment on such matters. There are a number of propositions in this part of the decision that certainly will not stand.

At page 457 the Court says:

"It is also to be noted that the depreciation in question is not that which has been overcome by repairs and replacements, but is the actual existing depreciation in the plant as compared with the new one. It would seem to be inevitable that in many parts of the plant there should be such depreciation, as, for example, in old structures and equipment remaining on hand. And, when an estimate of value is made on the basis of reproduction new, the extent of existing depreciation should be shown and deducted. * * * And when particular physical items are estimated as worth so much new, if in fact they be depreciated, this amount should be found and allowed for."

If this quotation means anything, it would indicate that the depreciation of the theoretical, estimated kind, which is normally "overcome by repairs and replacements" is not the kind of depreciation being considered by the Court, but only "actual existing depreciation." This is shown, for example, by the deduction for value of "old structures and equipment remaining on hand," after having served their useful life, and held, perhaps, for re-sale as scrap, or retained at full value in its books, by the corporation hoping to be allowed thus to increase artificially the "fair value" of its property on which the rate of return is to be allowed. In the use of reproduction new, "the extent of existing depreciation should be shown and deducted." This repeated use of "existing depreciation," and the word "actual" certainly indicates that class of depreciation which is in evidence, apparent to the inspector, and capable of ascertainment by examination to prove that it is "actually existing." The Court, further, expressly states that when "physical items are estimated as worth so much new," depreciation is to be deducted, "if in fact they be depreciated," and presumably not otherwise. In the very nature of physical property, it is impossible after it has once been installed that full 100% can be represented by all the parts, except for the service intended.

In the foregoing quotation from the Supreme Court's decision, the statement is made that depreciation must be deducted on the particular physical items being considered "if in fact they be depreciated." It will be noted that the Court expressly raises the question as to whether or not there has been depreciation, by the statement "if in fact they be depreciated." Absolutely, in every case, if market value —regardless of value for service purposes—were considered, depreciation of physical property would commence the instant it was installed, and there could be no "if" in the case. The use of "if" by the Supreme Court clearly indicates that depreciation should be deducted from reproduction new, only in case depreciation is obvious, or if the quality of service rendered has been impaired by use or by exposure. Mr. Floy.

If there is one thing that the decision of the Supreme Court in the Minnesota rate cases clearly establishes, it is that unwarranted assumptions, speculative bases, and hypothetical conclusions, will not be accepted as facts by this Court. This expression of opinion runs all the way through the decision, and clearly indicates that the determination of value by artificial rules, formulas, and speculation, instead of by examination of conditions, ascertainment of facts, and positive testimony thereon, will not have weight or carry conviction with the Court.

As the Supreme Court is unwilling to accept artificial rules, formulas, and speculation in determining value, as proposed by the Committee's Equal-Annual-Payment Method, and has repeatedly reaffirmed its decision that the present value of real estate is to be allowed, it will be seen that the recommendations of the Committee in these matters are inapplicable and misleading, and should not be endorsed by the Society.

The Committee apparently fails to appreciate the fact that the value of land is a capital account, not an operating credit or debit. If investors are to be allowed none of the chances for gain under advantageous circumstances, and must stand those losses accruing from disadvantageous conditions, the rate of return must be increased, but this is very difficult of accomplishment. If the investor were guaranteed, under all circumstances, a fair return, there might be some merit in the argument that he is not entitled to profits from the unearned increment, but such conditions do not now, and certainly will not, obtain in the United States in the near future; consequently, valuations proposed to be based on such premises are impracticable. If the utility investing in real estate is not to be allowed the unearned increment, the result will be ownership by individuals who will lease the required real estate to the utility at increasing rates, corresponding to the present value of the land, so that, in any case, the public will pay a fair return on the present value of the land.

The existing desiderata in utility valuation and accounting are directness and simplicity. Strict cost of reproduction applied to all

Mr. Floy. properties dispenses equal justice; has been approved by the highest Court; and is direct and fairly simple in its application when used by fair-minded engineers. It has come to be generally recognized as meaning the cost of reproducing new, a definite, existing, physical property on the basis of prices current at the time of estimate—and such estimate is made up to include everything that can be inventoried —regardless of original cost, age, service value, or present condition, as affected by depreciation. Reproduction new has not been accepted as meaning the estimate of cost at present-day prices, under conditions contrary to those existing at present and based on historic conditions. The Committee may consider that a combination of methods, namely, reproduction cost new and original investment combined, may be a fair method of getting at the existing fair value, but, for such method, the Committee should originate a new term and not attempt to legitimatize their innovation by adopting a generally accepted and widely understood phrase. If "cost of reproduction new," as defined by the Committee, is applied to long-established utilities, the resulting appraisal would not only be contrary to the decisions of the Courts, but would result in confiscation of private property.

When once determined, the cost of reproduction furnishes the basis for a uniform fair return to the investor, as well as the amount to be used in estimating the regular yearly allowance to be included in the rate charged for service to be paid by the consumer in order to protect the investment and maintain the service. The method of handling this latter payment to cover depreciation is a matter of detail, which can well be left to the public utility commissions, or other regulating bodies, but is not involved in the question of determining the present value of properties not heretofore under Commission rule.

It must be recognized that the investor in utilities has not put his money in to have it paid back in driblets, as proposed by the Equal-Annual-Payment plan of the Committee. The investor has placed his money with the expectation of a uniform return therefrom, on the part of the utility in which he has become interested; it is unfair and irrational to ask that he now receive back this investment in small, annual payments, which must be placed elsewhere, in order that the investor may receive the fair return on the amount which he considered his original investment in the utility. The gratuitous assumption of the Committee that, under its plan, the investment will be paid back yearly only in such amounts as may be required for additions to the property, is so highly theoretical, and contrary to what will be found practical, that such scheme cannot be seriously considered in the attempt to determine a generally acceptable basis of valuation and return between the utility and the public.

Mr. Whitten's book, which was of such great aid to the work of the Committee, is a masterful collection of Court and commission de-

cisions. Mr. Whitten, however, like most writers, has been unable to avoid all errors, as, for example, his conclusions with regard to the Court decisions in the Cedar Rapids Gas Light Company case and the Des Moines Gas Company case. His personal views and opinions are sometimes "against the clear weight of authority," as stated in a recent brief of a well-known attorney. Because of the evident dependence placed by the Committee on Mr. Whitten's book, the following quotations from a review of that book by the Secretary of the Committee is of interest.* Mr. Floy.

"* * * The author is well qualified for the work undertaken by him, which has been done fairly and without apparent bias, though in his expressed personal opinions the author leans to the public rather than the corporate point of view in his constant tendency to revert to original cost rather than present value as the most important yard stick for valuation.

* * * * * * *

"With reference to the cost of reproduction the author concludes:

"'Considered from all points of view the method of reproducing the existing plant under the actual physical and other conditions under which it was actually constructed seems fair to both parties. It is a rule that corresponds to the actual equities of the parties, while the other rule gives an unfair advantage, in some cases to the public and in other cases to the Company.'

"Here speaks the exponent of the public who seeks to enforce the doctrine of valuation by *original cost* rather than by the legal rule of *present value,* advocated by the corporation, and in force in all private and general business transactions. In the light of the past trend of court rulings, would not a fairer basis of determining reproduction cost involve the reproducing from the old original conditions of the plant in its present condition? The author's definition involves the reproducing of the work actually done by the company, not perhaps of the existing structure.

"The author's discussion of 'actual cost,' the possibility of its accurate determination, and its utility as a standard of valuation is certainly not justified by conditions in many of the older communities of the East.

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"In the discussion of 'Pavement over Mains' crops up again the strong bias of the public-service commissions in favor of original cost, rather than present value, as a standard of valuation. If the reader will constantly ask himself the question, 'Is this original cost or present value?' in reading the author's own comments and quotations from no less an authority than Commissioner Maltbie of the New York Public Service Commission, in the discussion upon the paving question contained in paragraphs 168 and 169, he cannot fail to be struck by the inconsistency of the position taken. It is unfortunate, too, that the issue should be clouded by any reference to ownership of the pavement.

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* Review by Leonard Metcalf, M. Am. Soc. C. E., "Valuation of Public Service Corporations," by Robert H. Whitten, *Engineering News*, September 12th, 1912.

Mr. Floy. "In the discussion upon 'Going Concern as Created Income,' the same strictures apply. The author prefers the Development Cost method of determination—clearly a reversion to original cost rather than present value. The author could not have been guilty of such a statement upon the reproduction cost method of determining going concern value as that contained in § 586:

" '* * * A public water supply is a necessity. A light system is a necessity. They are utilities that cannot be permitted to stop. The consumer must use them or suffer irreparable injury. *It is fanciful to speak of a development period for the new plant under these conditions.* If the old plant were wiped out, the new plant would have the business at once if already constructed, and otherwise, as soon as completed and ready for operation,'

"had he studied the actual experience of New Orleans, which has recently built new works, taken over an old small local plant, and in spite of drastic legislation requiring the abandonment of cisterns, is, in fact, *not in fancy,* slowly developing going concern value at substantial cost. So, too, his dicta upon going concern value:

* * * * * * *

"While the critic has no brief for either method and recognizes the inherent difficulties of the problem, the reproduction method of determining going concern value is the outcome of an honest effort on the part of the engineer to meet the requirement of the courts that the true reproduction cost shall be determined, and with all its possible inaccuracies, it is the only method yet devised which in any sense approaches such a determination."

The Committee has referred to the speaker's book, "Valuation of Public Utility Properties", using a quotation therefrom, as showing "the fallacy of the views expressed by engineers of much experience in valuation, that rates based upon the depreciated value of a property are necessarily lower than those based upon its full value." (Page 47.) As additional quotations from other writers are also given by the Committee, but without mentioning such writers by name, the exceptional honor thrust upon the writer would seem to be a little invidious. However, the quotations are from writers of such standing that to be classed with them and the Supreme Court in the condemnations of the Committee, offers no serious cause for complaint. The illustrations used by the Committee may disclose a fallacy in the methods proposed in its report, but they do not disclose any fallacy with regard to the practical matter of what is actually being done to-day in the valuation of utility properties and the fixing of rates based on such valuations. There are too many instances, known to those conversant with the subject, to need any citation here, in order to prove that the present theoretically depreciated value, where used for the purpose of fixing rates, results in a much lower return to the investor than if the full investment or the reproduction value new of the property had been

used for such basis. Therefore, the speaker's illustration does not seem to disclose anything in the way of a practical fallacy. Mr. Floy.

The soundness of the speaker's views as to the use of present value of land, inclusion of pavements over mains, and definition of cost of reproduction new, seemed to meet the approval of the Secretary of the Committee, at least, within the past year and a half, as shown by the following:

"The work of the commissions in determining land values is discussed in detail, as is the much debated question of the inclusion in or exclusion from reproduction cost estimates of the cost of paving over trenches, laid by the city subsequent to the construction of the pipe system by the corporation. The author points out the illogical position taken in this matter by some of our courts and commissions, and while admitting that in certain cases reproduction cost may not be a proper criterion of value, he soundly holds that in making an estimate of reproduction cost, the actual conditions of reproduction must prevail and the cost of the paving work be included, therefore, although not originally incurred by the corporation, as 'there exists no judicial pronunciamento for taking a part of the estimated cost of reproduction and leaving out the other part.' "*

The report (page 32) states that it is necessary to "adopt some practicable and equitable method of determining depreciation allowances" to meet certain requirements, which are given by the Committee. These stipulated requirements can be met much more simply and satisfactorily than by the adoption of the Equal-Annual-Payment Method. There is nothing in the requirements given by the Committee, which would prevent their being met by the use of either the Straight-Line, or the Sinking-Fund Method; which of these methods is preferable, on account of other considerations, need not be here considered.

One definite purpose of the Committee is apparently to provide equal annual payments by the rate-payers, continuously; and yet, as shown by the table on page 34, these payments are not equal, unless the same rate of depreciation and return is used. In order to attain this feature, which, in the minds of the Committee, seems so desirable, one of its members in a recent paper† states:

"I should like to suggest that the interest be the same on capital and depreciation allowances and that profit or wage be computed on cost of operation."

In order to have the report of the Committee acceptable to utilities, any requirement, that the rate of depreciation and rate of return should be the same, must be waived, and, as the greater the difference

* Review by Leonard Metcalf, M. Am. Soc. C. E., "Valuation of Public Utility Properties," by Henry Floy, *Engineering News*, September 12th, 1912.

† "Suggestions for Public-Utility Rate Making," by W. G. Raymond, M. Am. Soc. C. E., *Engineering News*, March 5th, 1914.

Mr. Floy. between these rates, the greater the inequality between the early and later years, during which the rate-payer is making payments for the service rendered, the recommendations of the Committee with regard to the treatment of depreciation must fall of their own weight.

Without discussing minor inconsistencies and other details with which the speaker cannot agree, it would seem as if the Committee was attempting to determine only one basis of "cost," which it considers should be used as the "value" of a given property. The Supreme Court requires that several bases of cost should be considered, and, by an act of judgment, the fair value then determined. It is on this account that the Committee's recommendation, mixing original-cost and original-investment methods, with reproduction-cost-new methods bring about a result which does not conform with any one of the valuations recognized by the Supreme Court. Such confusion is the cause of the Committee's conclusions that unless expenditures (many of which are not now ascertainable) have actually been made in connection with original construction, allowances therefor should not be included in the present value (page 28).

The recommendation of the Committee (page 43), classifying depreciation of property having a life of less than 5 years, as maintenance, and that having a life of more than 5 years, as replacement, is not progressive or in accordance with the best practice. The practical difficulty of separating maintenance from renewal accounts is generally recognized, and some utilities, with the approval of public regulating bodies, are now setting aside annually a lump sum to cover maintenance and renewals. Against this gross sum, the current year's maintenance and renewal expenditures are first charged and the balance of the appropriation is carried forward as a depreciation fund, for future renewals, thus simplifying the accounting, avoiding controversies between the representatives of the utilities and those of the regulating bodies, and insuring that the public receives full credit for all payments on account of depreciation.

The Committee proposes to depend mainly on calculated theoretical methods in determining depreciation, which do not give due regard to the condition of maintenance and repair in which the property may actually be found—regardless of its age. The Committee argues that the utility is only entitled to such allowance or depreciation as may have actually been expended (page 42), apparently failing to distinguish that the rate allowed for depreciation is entirely independent and distinct from the state of depreciation in which a property may be found to exist. Maintenance expenditures may almost wholly, if not entirely, offset losses from deterioration. Depreciation is but the sum of maintenance and renewal, and sufficient may be spent for these accounts to equal the full amount of deterioration.

Mr. Hendricks.

V. K. Hendricks,* M. Am. Soc. C. E. (by letter).—It seems desirable that the Committee should discuss more fully in the report the treatment of the enhancement in land values. As the writer understands Paragraph 19, Appreciation, on page 73 of the report, the public service corporation is to repay to the rate-payer the amount of the enhancement in values of land. As brought out in previous discussions, all other owners of property enjoy the benefit of such enhancement without having to pay for it, and the writer does not see why public service utilities should not be treated in the same manner.

Even if it were proper that the corporation should pay for the enhancement in value of land, there seems to be no way by which this payment could be made to the rate-payer at the time of such enhancement. Changes in land values are erratic, and there is no uniform change on which it would be possible to base any reliable calculations. If the public were paid for this enhancement by direct donation to the State or Federal Government, the payment would not be made to the rate-payer, but would be made to the general public. Such an adjustment would necessarily have to be made periodically, after the enhancement is known, and would be in the nature of a construction charge (or credit, as the case may be) and not in the nature of a refund to the rate-payer, or reduction in the rate.

As the writer sees the matter, the public service corporation should be entitled to the full benefit of enhancement in land values, and, even if this view is wrong and the Committee's view is correct, it seems impracticable to treat such appreciation "in the same way that the depreciation of the perishable property is treated, but on the opposite side of the account."

Mr. Wilgus.

William J. Wilgus,† M. Am. Soc. C. E. (by letter).—Since the preparation of the previous remarks by the writer, the Committee has issued a supplemental report, which is summarized in the following paragraphs:

"It has already been shown in the foregoing discussion that, in connection with the Replacement Method and the Sinking-Fund Method of providing for depreciation, value new must be used in order to do justice to the owner of the property, and, for the reasons given, it would obviously be confiscatory if the value in such cases were to be depreciated from year to year.

"In connection with the Straight-Line Method of providing for depreciation, it has been shown that equity requires that the depreciated value be used. When this method is used, the rates necessarily decrease from year to year, the amount of decrease being small in the case of short-lived property and large in the case of long-lived property, but this decrease does not represent injustice to the owner of the

* Previous discussion by Mr. Hendricks appeared in *Proceedings* for February, 1914, on p. 350.

† Previous discussion by Mr. Wilgus appeared in *Proceedings* for February, 1914, on p. 366.

Mr. Wilgus. property, as he will receive rates which, taken as a whole, during the life of the property, will be equitable. By the use of this method, he would receive too high rates when the property is new and too low rates when it is old, with a correct average amount.

"The Equal-Annual-Payment Method has been devised to obviate this difficulty, and when it is used in connection with the depreciated value of the property, as it must be to produce equitable results, the rates will remain constant regardless of the age of the property or the extent to which it has depreciated."

On railroads, except in the case of equipment, the restoration of depreciation of each item of property as it goes out of use is charged to current expenses, or, in some instances, to profit and loss surplus which may be looked on as the equivalent of a sinking fund; and therefore the methods referred to in the first paragraph of the quotation would apply thereto. Equipment renewals in part are charged to reserves, for which no standard practice prevails, some roads adopting very low rates and others comparatively high ones, all depending on the character of maintenance and the viewpoint of each company.

Utter confusion would follow any attempt to use both of the first-mentioned methods, and the third is believed to be inapplicable to railroad practice. Hence, the first method, of which cost-of-reproduction-new is the basis, is believed by the writer to be the only practicable and logical one to adopt, ordinary current depreciation chargeable to expenses to be defrayed out of current earnings, and deferred depreciation likewise chargeable to expenses to be provided through an allowance in the rate that will fairly compensate the owner for his assumption of the liability of making good such depreciation, as the need arises, without swelling the capital on which the public is expected to yield a fair return.

It is to be hoped that the report of the Committee, in its final form, will bring out clearly and forcibly its later views, which recognize that confiscation would follow the deduction of depreciation from cost-of-reproduction-new where the rates, coupled with the accounting regulations of the Interstate Commerce Commission and other regulatory bodies, do not provide for a gradual restoration of capital to the owner.

Mr. Dana. RICHARD T. DANA, M. AM. SOC. C. E. (by letter).— The great weight which this report will have in the development of valuation practice, and in legislation connected with it, makes it of importance to eliminate, as far as possible, contradictions in terms and ambiguous interpretations of the fundamental facts that must govern any theory of this subject. Valuation is the subject of the report, and there are at least several kinds of value, yet the Committee seems not to have considered it necessary to define the term, value, which it has used in several senses. Most laymen fail to appreciate the fact that confusion

in the use or meaning of this word is far more likely to result in injustice, or inequity, to the owner of a public utility plant than to the public, as will appear in the following. Mr. Dana.

Just as it has always been standard practice among engineers "to give the contractor the benefit of the (reasonable) doubt", so should it be the duty of appraisers to see that the owners of property receive rates which shall afford them a reasonable return on their property, and as long as the community presumes to substitute its own authority for the law of supply and demand, where free competition is not possible, just so long should it lean, if any way, toward the owner, in the determination to guard him against loss of his property by inadvertence and "without due process of law". It may be objected that the rights of the public must be conserved, which is true; but it is a common-law principle, centuries old, that before depriving a man of his property or of his liberty, the burden of proof is always on the community or the plaintiff. In a criminal action the State must prove its case beyond a reasonable doubt, but in a civil suit the plaintiff need only prove it by the weight of evidence. In these matters of rate-making the same broad principle of common-law justice, which for hundreds of years has maintained the sanctity of individual rights in all community matters, must sooner or later crystallize the practice into the same general form. The burden of proof should be on the community in all matters that are open to serious doubt as between it and the individual or corporation. The trend of practice is now plainly in this direction, yet many errors have been committed through failure to observe this rule.

The universal method in valuations is by synthetic estimates, in as great detail as possible. Now, the great fault of this method—perhaps its only fault—is that the likelihood of omitting items which should be included is far greater—a hundred times greater—than the likelihood of putting in items which should not be included. Consequently, every appraisal, other things being equal, is more likely to be too low than too high. The inclusion of a considerable percentage for omissions and contingencies is an effort to compensate for this inequity. Consequently, in a standard appraisal it is reasonably certain that the net amount of the omissions, as reduced by the allowance for omissions and contingencies, will be in small proportion to the whole amount of the valuation; but because there is to-day great confusion as to the meaning of the term, value, an owner may be deprived of a very large part of his property without compensation. The Courts in general will allow a "fair return upon a fair value" (page 6),* and it has been held (page 20),* "that the value of the property is to be determined as of the time when the inquiry is made regarding the rates". The Committee in its report infers that this implies that present (as of the time of the appraisal), rather than original, unit prices. should

* Committee's Report.

Mr. Dana. be used in determining the cost of reproduction. Does this necessarily follow? The whole matter hinges on the meaning of "fair value", and the writer purposes to show that, with a logical definition of this term, the Committee's inference is untenable. The value of a plant is its intrinsic worth to its owner. Measured in money, it is "that sum which he could exchange it for without intrinsic gain or loss". As ownership exists for the purpose of periodical return (interest or dividends), the theoretically perfect method of computing intrinsic worth would be by capitalizing net earnings, but this is impossible for rate-making purposes, involving reasoning in a circle, as has been often pointed out.

The application of the definition of value just given affords a convenient criterion for testing the method of computing unit prices. By way of illustration, assume a private water plant, 60 years old, with a large reservoir excavated originally in earth. The earthwork cost probably 40 cents per cu. yd. by hand methods (old rates of wages), but could be done to-day for 30 cents (steam shovel and present rates). If the owner should exchange this reservoir for money on the basis of 30 cents per cu. yd. for the excavation, he must stand a loss of 10 cents per cu. yd. It cannot be objected that the cubic yard which originally cost 40 cents to remove has depreciated "functionally" or "naturally", unless another reservoir could be dug for less cost or for the same cost with greater usefulness. It cannot be objected that he lost the 10 cents per cu. yd. as soon as the steam shovel was developed, and that the community is not responsible for what he lost before the appraisal, because he has not lost it until he parts with it, as long as it is useful to him in his business. If 10 years or 2 years before the appraisal the reservoir became useless, then at that time, he made a loss of the full cost, perhaps. If it deteriorated functionally, then there was a partial loss by depreciation; but with such functional depreciation, which depends on capitalized net earnings of the depreciated plant in comparison with those of a standard plant, the steam shovel has nothing to do. The cubic yard which was taken out by hand 60 years ago is bringing him the same income as if it had been excavated with a steam shovel. As it could not have been excavated with a steam shovel in those days, its actual cost represents the fair equivalent of what the owner has invested in it. If he receives less than this for it in exchange, he stands a loss. If he receives more, he gets a profit. If the community allows him 30 cents for it, it compels him to lose 10 cents, and deprives him of his property without compensation. He might be willing to sell it for 30 cents and take a 10-cent loss in order to go into a more profitable business, but the community has no right to make him sell it and take such loss.

From another viewpoint, consider a telephone company which owns, among other things, 100 000 000 lb. of copper wire on poles, for which it paid a weighted average price of 15 cents. Its investment in the wire will then be $15 000 000. Now, suppose that at the time of ap-

praisal the price of copper wire was 29 cents. If allowed rates on the basis of 29 cents, the company would get a profit of $14 000 000 on that item. It cannot be argued that the company made the profit as soon as the market changed and the open market price rose to 29 cents, as it reaped no benefit from the market rise if it could not sell its wire, but kept on telephoning over it. A profit sans benefit is of infinitesimal value. The company would have made a profit if it had sold at 29 cents, but had it done this, it would not have had the wire over which to telephone. The community, in taking over this wire at any other price than that which was paid for it, must either compel the company to take a loss or allow it to make a profit, neither of which alternatives is consistent with a fair "value". There will be some copper wire in the warehouses, say, 10 000 000 lb., which is not in use, is not earning anything, cost 15 cents, and could be sold in the market for 27 cents net. It does not have to be kept on hand for emergency repairs, but is held pending some proposed new construction. It differs from the wire on the poles in that it is exchangeable for cash at 27 cents without interfering with the operation of the company as a going concern. Its owner can sell it the day before the appraisal and "take" a 12-cent profit. This he cannot do with the wires on poles. If the community allows the company 15 cents for the wire in storage, it will compel it to part with the profit which it earned by speculating in copper (which is what it did when it bought in advance of its wants), and, therefore, it would be causing the company a loss on the exchange, which is inconsistent with the definition of a fair value. To avoid intrinsic gain or loss, the community should allow original unit prices for items which are not marketable by the owner without going out of business; and market sale prices for such items of plant as can be disposed of without interfering with the business. A corollary to this rule necessarily must obtain when a loss has taken place or a profit has been earned on plant in service. Mr. Dana.

It is possible for the owner to suffer a loss or gain a profit on the plant without having a free market for it. Depreciation entails a loss to the owner, and when it is a function of the lapse of time (natural depreciation) to avoid this loss, it must be compensated for, either by a sinking fund, to increase *pari-passu* with the depreciation, or by an equivalent amount invested in some other manner, such as other plant (renewals), etc. On appraisal, the depreciated items always entail a loss which was incurred by the owner before the appraisal and with which the community has no concern, as such loss is not a result of the appraisal. If the owner has kept his investment intact by a depreciation fund or by purchasing other plant or investing in some other manner the equivalent of the depreciation sinking fund, then the value of the total assets remain as they were, but the part of the plant which has depreciated has already stood the owner a loss. The community must allow its depreciated value, in rate-making, else it would be

Mr. Dana. standing part or all of the loss already incurred by the owner during the years of operation, which is against the hypothesis in the previously given definition of value. In relation to this matter, two points must be very carefully noted: First, that natural depreciation is a loss of value, proceeding gradually, which cannot be determined by market conditions, nor even by inspection of the plant, but must be calculated theoretically on certain assumptions of life, age, and interest rate; and second, that the capital may be preserved intact by a separate depreciation fund to be set aside (a very rare occurrence in practice), or by investing an equivalent amount in other plant or in some other asset, or again by a part of the potential increase of business caused by the money spent in development (or not paid in interest or dividends during the early years of the business) and which has been termed "development expense". Considered in this manner, there is no difficulty about constant rates and constant dividends, without impairment of capital and without departing from a consistent definition of value. If the owner has foolishly wasted money in accordance with bad practice, he is not entitled to have it covered by development expense, as it is not a proper expense of development.

There remains to consider functional depreciation. Consider a 30-year, single-cylinder, non-condensing, steam engine belted to a modern generator. Assume the engine to be in perfect condition and functioning as well as on the day on which it was first placed in service. It is transforming steam power into mechanical work at a certain rate and at a certain unit cost for that function. Nothing has been spent on it in that time except for care, housing, and ordinary repairs. It could not be sold except for scrap. No fund has been set aside with which to buy a new engine in its place. A modern engine of the same capacity can be bought, which will transform steam power into mechanical work at a unit cost which is to that for the old engine, say, as n is to m. These unit costs do not include interest, etc., on the prices of the engines. The new engine could be bought and erected for N dollars. The amount which represents the fair value of the old engine is that figure which will make the unit cost of transforming energy by the old engine the same as that of transforming energy by the new engine, if bought. Otherwise, if the new engine can transform at less unit cost, the owner could exchange and make a profit; but, on the other hand, if M be such a figure that the interest charges and operating expenses of the new engine would be greater than for the old, a compulsory exchange would result in a loss to the owner.

Let the cost of the new engine, connected............ $= N$
" " value of the old " .... $= M$
" " unit cost of transforming power by the new engine $= n$
" " " " " " " " " old " $= m$
" " number of units of power transformed per year.. $= U$
" " interest rate on money...................... $= r$

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Then the annual cost of operating the new engine would be $Nr + nU$..(1)
" " " " " " " old " " " $Mr + mU$..(2)
" " unit " " " " new " " " $\frac{Nr}{U} + n$...(3)
" " " " " " " old " " " $\frac{Mr}{U} + m$..(4)

By the conditions of the problem these must be equal, and, therefore,

$$\frac{Mr}{U} + m = \frac{Nr}{U} + n, \text{ whence, } M = N - U\frac{m - n}{r} \quad \text{.......(5)}$$

It can be shown that this is the same value for M which would follow from equating the capitalized annual cost of operating the two engines.

The owner of a certain water plant thinks that it is worth more than $100 000 to him; the mayor of the city thinks that it is worth $50 000 to the city which wants it and has the right to acquire it on a valuation by disinterested experts. The city would scrap a large part of it and have an entirely different kind of works, and considers itself justified in allowing scrap value for all the parts of the plant that it would like to scrap, on the ground that the new plant would pay for itself in a few years out of increased net earnings. The city's adviser estimates that for a certain pump the value of M is actually negative. On this theory it would be consistent to allow the owner less than nothing for the "value" of his pump and of any other plant which, by this formula, will be less than nothing. The solution may be found by applying the criterion that "value is that sum which can be exchanged for the plant without intrinsic gain or loss to the owner". The pump is raising water which is being sold for more than the operating costs. The owner is receiving a return on the money which he has invested in the pump and the remainder of the plant, and he is now receiving some return on money spent in developing the business, by way of Development Expense. The pump in question is earning money, and it, or some other pump, is essential to enable the owner to operate the business. Manifestly, to take the pump away from him without paying for it would entail a loss to him. He admits that he could earn more by buying a new pump, but he has put so much capital into the development of the business, that he does not wish to invest more money at present. The money spent in development has been wisely expended, because the net earnings are now nearly enough to pay a reasonable return on all that he has invested in the water-works, and the net earnings are growing so rapidly as to promise in a few years a profit on the investment, when he will be in position to secure plenty of additional capital for improvements, etc. Now, the city does not contemplate buying from him just the pump

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alone, at a negative valuation. It has an option on the whole plant of which the pump is but one part, and he stands no loss if he receives a negative value on the pump, provided he gets a proper positive value in the form of Development Expense. If it should appear that he has received a fair return in dividends and interest on the capital invested to date, operating expenses to include depreciation, then there is no Development Expense, and the pump no longer represents to him a capital investment equivalent to its original cost.

In calculating the Development Expense proper depreciation charges should be considered a part of operating expenses, in arriving at the correct amount of net earnings. On that basis, and that basis alone, can the Development Expense be correctly figured, and then it is logical, consistent, and proper to allow to the items of plant their true functional value, which may be, and often is, negative. Where this practice is omitted, injustice is sure to be done. When properly applied, it affords the owner ample protection against actual loss, and, at the same time, enables the community to acquire the plant or fix the rates on a basis no higher than would have been the case had the community built and developed the plant in the same manner and at the same time that the owner actually did develop it. Moreover, as the development by the owner is likely to have been more economical and efficient than if it had been done by the community, there is a greater or less intangible profit accruing to the latter if the owner is allowed only the exact amount represented by his investment. To regard the owner as having been in effect the agent of the community in developing the property, besides being fair to the owner, is not unfair to the community. On this criterion is based the so-called "Agency Theory" of valuation.

Very frequently, and nearly always in the case of railways, it occurs that competitive rates are such and development expense has been so heavy that the owner is not receiving a normal rate of return on his capital, nor can he perceive such a return in sight. He, therefore, would be glad to sell his property for less than the "fair value", taking a loss in order to go into a more profitable business. If the community can ascertain what this amount is by bargain, then a purchase at that price may be effected, but obviously it would be the height of injustice to regulate rates so as to cut down the net return below a normal rate on the owner's capital in order to coerce him into selling at a loss. By the same token, his point of view of value is that which should be taken in fixing rates.

Aside from these reasons for using original rather than present unit prices in computing plant value, there are occasional difficulties in the practical application of the present-unit-price method. For instance, in Central New York State a good many piles, driven 50 or 60 years ago, are of the most magnificent white oak, with butts of

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22 in. and more. There is no market for that kind of material at present in that part of the country, and freight from a long distance would make the unit prices on such material prohibitive to-day. To apply present unit prices then on such material is virtually impossible, except by assuming that the owner should be contented with the unit price on something else "just as good". Consider the matter of ashlar masonry laid in Portland cement mortar when the cement cost $4 per bbl. To-day, the same function would be performed by concrete made with cement costing less than half that price. To allow the owner present prices on concrete and calculate his masonry in that manner, in fixing rates, would compel him to stand a very considerable loss. It cannot be objected that he must stand that depreciation anyway, because the original cost of the ashlar with the high priced cement must be depreciated to get the present (to him) value, as outlined. To allow present prices of cement and calculate the present reproduction cost of ashlar, would force him to take an unfair loss on the cement, and so little ashlar masonry is built in these days that there is hardly a free market price for it. It is in a class with the large oak piles previously mentioned.

The writer understands from the report (page 15) that it is the opinion of the Committee that though present prices should be used, the estimated cost should be based on the conditions existing at the time of construction. As prevailing prices are among the conditions of construction, the Committee's ruling seems to the writer to lack consistency, especially as

"The Committee believes it to be a general basic principle that where money has been expended with due discretion * * *, the company is, in the case of a newly erected normal property, entitled to a fair return upon a value which, subject to modifications, equals the amount of money invested."

With the last paragraph under "Unit Prices" (page 20), the writer cannot agree. In the case of lumber, as quoted for illustration in the report, the prices have been rising, and probably will continue to do so in the East, until the Panama Canal is finished, when they will go lower in the East and higher in the Far West. Can the building of the Panama Canal afford an excuse for cutting the rates charged by a railroad in Pennsylvania, which has been able to earn just enough to pay interest on its actual capital investment, with depreciation on plant which includes many timber trestles? To depend on the appraiser "to select the length of the period with a view to obtaining a fair prevailing price" does not seem logical or practical in its application. The writer ventures to call attention to the fact that, in the vast majority of cases, the aggregate depreciation is much more than the aggregate appreciation on physical property, and that this fact is likely to militate against the fair interest of the owner, unless a correct

Mr. Dana. amount for Development Expense be allowed. Now, this line of reasoning assumes that there is a positive "Development Expense", and is consistent with the "Agency Theory" of valuation, to which four very important objections (among others), have been made, *viz.*:

(1) That the owner never was an agent and, therefore, should not be considered as such;

(2) That the owner is entitled to appreciation in value, in such items, for instance, as real estate, arising from the development of the community in which his property is located, and that the theory outlined does not allow him such profits;

(3) That when the rate of allowable return is taken from the standpoint of the owner, the community is overcharged, because it could have obtained the money at much lower rates than the "fair-return" rate to the owner;

(4) That the Agency Theory eliminates Franchise Value.

(1) The first of these objections, which has been made in legal proceedings, is somewhat overstated. Had the owner actually been appointed agent for the community, the latter would have to stand for all his blunders, except where intent to defraud was demonstrable. Frequently, the community treats the owner as anything but its agent, as in the case of Seattle, Wash., holding down his rates by commissions and competing with him by a municipally owned plant. Whether or not the owner actually was an agent seems to be beside the issue, except on the basis of a purely legal technicality, which need not concern us here. The Agency Theory has been thus called because the term conveniently expresses a logical hypothesis which, in its proper application, protects the owner against actual loss, except by blunders, as though he were the community's agent.

(2) The second objection is of much importance. There are cases in which development expense has been extinguished by surplus earnings, although this is rare indeed in the railroad business.

Where the land can be sold and the profit gained without interfering with the business as an operating entity, it seems that the owner is entitled to the profit, as before discussed. In regard to the property which is an integral part of the business, the Courts have ruled that the owner is entitled to appreciation, he is taxed on the appreciated value, and if he be not allowed to earn a return on the appreciated value, what justification is there in making him take a loss on property which has been lost or has depreciated, such as an abandoned piece of track?

Were a possible competitior to appear and attempt to buy the owner out, there is little doubt that the competitor would expect to pay about the present value of the land. If he succeeded in getting it for less, he would consider it a fine bargain. If he had to pay much more he

Mr. Dana.

would buy elsewhere, or give up the idea. Now, the community, under the common-law theory of individual right, has no right to compel the owner to take less for his land than a competitor would expect to pay. If there is other land available, the community can buy the other land and allow the owner to use his land in some other business or otherwise dispose of it. It might not want to do this, because of the expense of moving tracks and buildings to another site, but that is no reason for not giving the owner his profit, if he is going to be forced out of business. In such a case as that of the New York Central Terminal on Manhattan Island, there is no other available site for the purpose, but here the owner must be entitled to a value based on the assumption that he may proceed to enjoy equally desirable land if he should want to go into some other business or otherwise dispose of his land, turning over the old business to the community. For rate-making purposes, the rates must be adjusted on the assumption that the owner would fare as well as if the community bought the property on the fair valuation. Any other view than the one stated will make the owner wish that he had used his property in some other business, one in which the community would have left him in peace to enjoy his prosperity. Emphasis should be laid on the fact that the "greatest good to the greatest number" is consistent with placing a premium, rather than a discount, on the use of private capital for public purposes. Any other view means socialism, if not anarchy.

(3) In regard to the third objection, the writer believes that if the owner put in his own capital, he has a right to the same return on it as if he put it into another business. The community might have borrowed it for less, but it did not, and as it had its choice in the matter and the owner did not, he is entitled to the normal rate of return on private capital.

(4) The fourth objection has certain intrinsic features in common with the second, and furnishes a clue to its solution.

If the owner is to be paid rates which will just allow him to earn a fair return on his investment, and there is no prospect of his being able, or allowed, to earn more than this in the future, of what good is a franchise to him? The original and only idea of a franchise or charter has always been to induce him to apply his money to public purposes rather than to use it in some other form of industry. Now that the State has money of its own, it seems hardly equitable to penalize the owner because he allowed himself to be persuaded to invest it under the inducement of a franchise or charter. Where there has been expense of development, which has not been amortized out of net earnings, and is not likely to be, there is no "franchise value" anyway, and the "franchise" simply amounts to a contract on the part of the community to let him stay in business and try to make up his losses. Where there is no accrued development expense a franchise

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is a public bonus, in the form of a contract, whereby the owner may expect to earn an extra return on his capital as an inducement not to go elsewhere with it.

These four objections to the Agency Theory are based on what has been termed the Competitive Theory, which sometimes gives a total appraisement higher than the Agency Theory, and sometimes lower. In general, where there is a considerable Development Expense, the Competitive Theory gives a lower total than the Agency Theory.

It is often necessary in an individual problem to make two or more valuations, one on the Agency Theory, and another on the Competitive Theory, in which case the totals will not agree. It is certainly illogical to make a computation of value, some of the elements of which are calculated on the one theory and the others on the other.

Mr. McCormick.

R. S. McCormick, M. Am. Soc. C. E. (by letter).—The Committee has presented a very able and exhaustive report, and it will doubtless be received as expressing the Society's stand on this subject.

The writer, however, desires to call attention to the fact that, in his opinion, the Society cannot afford to go on record in this way, and although the Committee has stated that the report is incomplete, and doubtless discussion will bring out points not touched on, he feels that it would have been more consistent with the object and aim of the Society to have confined the report to the question of valuation alone, omitting the application of such valuation, at least, to the extent of applying the whole argument to this purpose. The report might have been treated as an abstract subject of valuation, with and without depreciation.

There is a difference between the valuation of railroad property and other kinds of public utility property for rate-making purposes. Competition is not dead in the industrial world, not even as between large combinations, and although this competition is of a different kind from the old sort, it is not the less real, and its effect on the fixing of rates is a live issue between traffic managers on railroad systems. Where this kind of competition is possible, the matter of rates will never be wholly, or probably in any great measure, based on the principle of a fair return on the value of the property. This principle, however, is equitable, and, where it is possible to apply it, will give equitable returns. In the case of public utilities such as water-works, street railways, gas and lighting plants, etc., it is possible so to base rates. In the writer's opinion, however, it is wholly impracticable to apply this principle in this way to 95% of the railroad mileage of the United States; and, to say that these properties are valued for rate-making purposes is a misnomer.

The valuation of railroad properties, no matter how made, or on what principles as regards the question of depreciation, can be useful

only as a guide to prevent exorbitant rates, and can never prevent lower rates than will give a fair return on the investment. Mr. McCormick.

In the case of local public utilities, on the other hand, this valuation can be the fixed base for rate-making, because, speaking generally, there is no competition. For this purpose only a discussion on valuation for rate-making is properly applicable.

To value properly any one of the great railroad systems involves a most intricate and exhaustive examination by men properly qualified, and to know when an equitable value has been given to the properties is well worthy of the best efforts of those engaged in the work.

When all the data are collected and reported to the Commission, and a value is placed on the multitudinous items comprising this inventory, its value for rate-making purposes can be a fit subject for discussion.

The writer is of the opinion that such a valuation should be for the sole purpose of ascertaining the worth of the properties as property, keeping in mind the use to which it is put, and should have no bearing as to the purpose for which the valuation is made. This value may be present value, original cost of property to date, or a value based on cost of reproduction with or without depreciation. The various differences of opinion as to which method will give the true value presents a sufficient field for discussion without including the purpose for which it is to be made. It is unwise to go at this subject with any other object in view. This is especially applicable to railroad property, and the Interstate Commerce Commission has begun this work under a most serious handicap because the railway companies are getting a wrong impression in regard to its object.

It will not be possible to carry on this work and do justice to the carriers unless the engineers of the Commission and the companies' engineers agree on a common basis regarding values. To hamper either side with the burden of reconciling these values for any particular purpose is essentially wrong in principle, and, at least, should not be a subject for discussion until the inventory of the properties is complete and the value fixed by the Commission under the terms of the Act. Section 19-*a* certainly is explicit as defining what the Commission is to do, and it does not say that this valuation is to be made for the purpose of rate-making.

Although it is understood that such at least is one of the objects of the valuation—perhaps the principal one—the Commission has no authority to make this valuation with this end in view, and hence the Committee's report should have dealt with the subject in a broader sense, or, if this purpose—rate-making—was the object, it should have recognized the distinction between public utilities of the class where absolutely no competition is possible, and those where it is possible. This competition, although perhaps not fixing the rate, has, in a

Mr. McCormick. broad sense, much more to do with it than the actual value of the property could ever have. The physical value is of great assistance in adjusting rates, but not in making them.

Mr. McCarty. RICHARD J. MCCARTY, M. AM. SOC. C. E. (by letter).—Realizing the importance of light on valuation of public utilities, the Board of Direction in October, 1911, appointed a Committee to investigate and report on the subject. At that time the full significance of the problem had not been developed, and it was generally regarded as one to be solved in the main by the Engineering Profession.

To judge from its Progress Report, the Committee addressed itself to the task with the utmost energy, and did great credit both to itself and to the Society by the public spirit manifested.

The passage of the Federal Valuation Act, and its approval on March 1st, 1913, made the problem one of national importance, and caused its fundamental significance to be investigated by many who before that time had given it particular attention only as applied to special cases.

A thorough consideration of the general character of the problem shows that its ultimate solution must be reached from the standpoint of Economics, Law, and Equity, rather than of Engineering. That the Committee realized this is shown by the following quotation from its report:

"The principles and methods involved are now being carefully considered by many public service commissions and the Courts, as well as by engineers, lawyers, and committees of various societies having to do with public utilities. The subject is clearly in the developmental stage, and any conclusions reached at the present time must be considered as tentative."

The Committee then states that:

"It is recognized, however, that the whole subject is in a developmental stage, and while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

It is not here held that the problem of valuation does not call for the highest type of engineering ability, but it is held that the work of the engineer terminates at the point at which the quantities and values for use of the different parts of the physical property of any given public utility shall have been determined, and that from this point forward the problem ceases to have particular significance to him. The reason for this is that the determination of the value for use of physical parts of a public utility, and the application of such values in determining the amount of capital on which a public service corporation is entitled to a fair rate of return, are problems of an

entirely different character, and belong to entirely different departments of human thought and experience. Mr. McCarty.

This is recognized in the Federal Valuation Act itself, which requires that there shall be ascertained and reported the original cost to date, cost of reproduction new, cost of reproduction less depreciation, other values, elements of value, and, separately from improvements, the original cost and present value of lands, rights of way, and terminals. The Act, however, is so far from stating the manner in which this information shall be applied, that it does not even give the purpose for which it is to be used.

The manifest intention of the Act is to obtain certain information through the Engineering Profession concerning the physical features of public service utilities, and to use the results thus obtained as a basis for determining the amount of capital on which the owner is entitled to a fair rate of return, and for other proper purposes.

For the American Society of Civil Engineers to investigate and give to the Engineering Profession information and data concerning the determination of the value for use of the physical parts of public utilities is eminently fit and proper, but for it to go beyond that point would be an invasion by the Society of the respective provinces of Law, Equity, and Economics.

This, the writer is convinced, the Society cannot afford to do, either in justice to itself, as the leading engineering organization of the United States, or in justice to the public, which heretofore it has benefited to such a great extent.

The effect of an attempt by the Society to solve problems of Law, Equity, and Economics would be great, because of the great influence on the Governmental authorities, and on the Courts, of any conclusions which it might reach or apparently endorse, and inasmuch as the Committee itself had recognized that any conclusions reached at the present time must be considered as tentative, it naturally follows that such conclusions should not be advanced under the auspices of the American Society of Civil Engineers.

The writer, therefore, is of the opinion that the report of the Committee should be confined strictly to the determination of the value for use of the physical parts of public utilities, and should not make any recommendations, or advance any tentative conclusions, concerning the manner in which the results of such determination shall be applied to questions arising between the owner of the utility on the one hand and the public on the other.

It seems particularly important that the Society, as well as the Committee, should adopt this course, because, it might easily happen that, owing to the influence of the former, and the standing and reputation of the members of the Committee, certain tentative conclusions might be embodied in the decisions of the highest Courts, which after-

Mr. McCarty. ward might be shown to be untenable, and thus work great injustice either to the public or to the owner of the public utility.

Mr. Buel. A. W. Buel, M. Am. Soc. C. E. (by letter).—Has the Committee considered what items should be covered by a "fair return"? If it covers risk of investment, why not also risk of development expenses or deferred dividends? It is necessary to answer these questions, in order to avoid duplicating the same items in the valuation and in the fair return. Is there a method by which a "fair return" may be determined from statistics? Although the final answer to these questions will have to depend on decisions of the Courts, it is the duty of valuation engineers to collect and present all data relative to the discussion.

Losing ventures would probably occur as often, if not oftener, under public ownership, and, therefore, they should be given some consideration. They may be divided into two classes with such approximation as can be ascertained by the historical facts. First, those which at their inception appeared to offer fair hopes of becoming profitable within a reasonable time, such as would attract conservative capitalists, but which failed to develop as anticipated; and, second, those which, at the time they were promoted, would not have appealed to prudent men as reasonable business risks. If the second class be eliminated, and the first class of losing ventures be combined with more or less profitable properties of the same kind, similarly conditioned, the expenditures for capital account and the net earnings or operating incomes for a series of years, for the entire group, should furnish a basis for determining the percentage of risk involved.

All the capital invested in the group, less the capital losses divided by the capital invested, gives the percentage of security. If a fair return, where no risk of loss is involved, is assumed at 4%, to compensate for the risk of loss of capital, it must be increased to the percentage that 4 would be on the percentage of security. Thus, if an average of 20% of capital invested in railway construction were lost, the percentage of security would be 80, and 4 divided by 80 is 5%, the return required to compensate for the capital risk.

To find the "fair return", however, we must also add compensation for lost and deferred dividends, unless these have been added to the cost of the properties in the valuations as development expenses. It would seem preferable to take care of these items as elements of risk in fixing fair returns, rather than to add them to cost of property as development expense, because they may vary greatly in the same territory and on competing lines which, both on the ground of equity and of the practical difficulties involved, require equal treatment on the common basis of the average return on the total capital investment in the group plus lost or deferred dividends, excluding losing ventures of the second class. It seems quite as logical to treat them as risks

or losses as to consider them as development expense added to valuation costs. This method also has the advantage of greater clearness in segregating the values of purely physical property, thus reducing complications in arriving at operating values or valuations as going concerns, or, as some call it, good-will. Mr. Buel.

If we add all lost dividends, compounded at 4%, up to the average age of operating maturity, at which profitable properties begin to earn fair returns, to the capital invested in the group, then 4% on this sum divided by the percentage of security gives the fair return for this group.

Assuming 10 years for the age of maturity, for example, a percentage of security at 80 as before, also that the 20% of losing ventures of the first class just earn operating expenses and taxes on the average, and that the average earnings of the 80% of paying properties during the first 10 years is 2% less than a fair return, we have:

Lost dividends on 20% of losing ventures, compounded at 4% for 10 years, amount to 20% of 48.02%, or......	9.6%
Lost dividends on 80% of paying properties during 10 years of development at 2% compounded at 4% amount to 80% of 24.01%, or.....................	19.2%
Total lost in dividends.......................	28.8%
Original investment in group.....................	100.0%
Total " " "	128.8%

Four per cent. on 128.8% is 5.152%, which divided by 80 gives 6.44% as the fair return that insures an average yield of 4% on the investment. If this group represents four competing railway lines, the average return of 6.44% might be made up of 2% on the investment in one line, 10% on another line, and intermediate rates on the other two.

Of course, it is understood that these figures are merely assumed for an example, but it ought not to be impracticable to derive fairly close percentages for the losing ventures and lost dividends from a careful analysis of the records of large groups of kindred properties. It would probably be necessary to fix some limit on the age of maturity.

If the percentage of security is 90 and the lost dividends compounded at 4% amount to only 14.4%, then the fair return would only be 5.08 per cent. Of this, 4% may be called interest on investment and 1.08% insurance of capital and interest.

Though the Committee recognizes the equity of some recognition of operating values, or, to use its term, "efficiency", as a "modification of the physical valuation or of the rate of return upon such valuation", it in effect puts this problem up to the commissions

Mr. Buel. and the Courts. Without question, some points involved in this element of valuation will have to await Court decisions for final determination; but, on the other hand, the larger part of this problem consists of highly technical matters which will devolve on engineers to solve. In fact, the determination of operating values or efficiency seems to be more especially an engineering problem than one of depreciation, which is largely a matter of accounting, or of inventory valuation of the real estate and many other elements of the physical property. The Committee's report suggests, quite correctly, the great difficulty of formulating any general rules for the practical application of this feature, but, because of its importance and the fact that it is generally so little understood, it deserves particular attention.

This problem of efficiency or operating values is so complex that it is extremely doubtful if it will be possible to lay down any formulas or rules except those of the most general character. Possibly, the only practical method will be found to consist in making comparative estimates of operating values of the existing property and of an ideal or hypothetical equivalent on some revised location or design, which involves surveys for revisions or new designs. But intelligent reconnaissance work will confine the extent of such investigations to practical limits. This method is not only feasible for determining operating values of railways and other properties with sufficient accuracy for all practical purposes, but it can also be used to advantage in estimating the efficiency of operating methods. Estimates of operating values and operating costs should be made in full detail, treating each variable item of cost separately for each varying condition. This is laborious and voluminous, but, for many cases, it is the only convincing method which as yet has been suggested. The writer has used it in studies of several thousand miles of railway locations and revisions under widely varying conditions, and it has always proved satisfactory. The degree of accuracy that may be attained by this method depends principally on the care used in finding correct data, particularly of the unit costs, and the formulas by which they vary. The latter, of course, can only be arbitrary approximations in most cases, but, if the data are based on broad averages taken from statistics of actual operations of properties working under as nearly similar conditions as possible, the final results will be substantially correct. No good would result from attempting a greater degree of accuracy than the variation of one normal year or another from the average, and the method suggested can be made to approach this if care is taken and good judgment used.

As the efficiency of any plant, location, or design depends to the greatest extent on its proper adaptation to the volume of business handled, it would be decidedly unfair to leave out of consideration the volume of business in sight, or that could have been reasonably

foreseen, at the time the property was constructed. The volume of business may have increased to such an extent that the original property or parts of it are practically obsolete, requiring reconstruction, revisions, and new facilities. No doubt many questions will arise as to just what is the equitable way to handle such cases of insufficiency of plant, but the circumstances and conditions may be so variable that a general rule could not be formulated to cover them. They probably belong in the class calling for the use of the best professional judgment available, except where they may be covered by Court decisions. Mr. Buel.

In another class are a few cases in which expensive revisions or reconstruction have been undertaken, entirely unwarranted by the volume of business in sight. For example, a railway with comparatively light traffic may have spent many millions of dollars in revisions to reduce mountain grades, without cutting down the summit elevation and at the expense of considerably increasing the distance, the savings in operating costs, being entirely inadequate to pay the interest on the additional investment. While, in time, the traffic may grow to a volume warranting the revision, if the expectation of such a result is deferred to an unreasonable time the revision should have been deferred. In such a case it is questionable what, if anything, should be allowed in the valuation for the cost of the revision. Though such cases are probably rare, some will be met. The example suggested serves to bring out by contrast the more numerous classes of poor original location or design and of those which have become obsolete and inadequate due to the growth of traffic, all of which suggest the large number of important and difficult questions in valuation still remaining to be solved.

J. E. WILLOUGHBY, M. AM. SOC. C. E. (by letter).—The subject under investigation of the Committee was worded so as to give prominence to "railroad property". It seems unfortunate that the report draws its illustrations principally from, and is pervaded with ideas relating to, water-works and sewerage plants. It is possible that the treatment would have been somewhat different had the underlying idea of the Committee been the valuation of railroad property. The writer's experience is confined to the construction, maintenance, and valuation of railroad property, and the comments hereinafter made are formed from experience with railroad property only. Mr. Willoughby.

The writer is not in sympathy with the formulation of principles and methods of valuation for rate-making, as distinguished from any other purposes. Whenever such a distinction is made, the presumption is raised that a valuation for rate-making purposes differs from, and is higher than, a valuation made as the basis of a sale. Such a presumption is unjust to the owner. The items which enter into the valuation covered by the Committee's report appertain almost exclusively to the determination of the cost (either original or replace-

Mr. Willoughby. ment) of the physical property, and a valuation confined solely to the cost of the physical property is unjust to the owner. For example, take good-will. In a water-works monopoly, good will may be of but small value, but in a railway property it is often that which makes one railroad prosper, while the competing railway goes into bankruptcy. Inasmuch as the valuation of a public utility property includes other items than the cost of the physical property, the Committee's report is incomplete, and in that form it prevents the assignment of losses due to use (*e. g.*, depreciation) to the proper place in the valuation.

General Principles Involved in Valuation for Rate-Making.—As a corollary to the first of the two features suggested by the Committee as to the requirements of justice in any regulation of rates (page 7), the writer suggests that the annual return, in addition to covering interest and profit for the use of the capital, should provide such earnings as will enable the owner to meet the demands of the public through its public service commissions for additions to the property which cannot be capitalized without affecting the general credit of the owner. A familiar example of this kind of additions is found in the rivalry of the cities, and even towns, for union stations, monumental in character, and costing out of all proportion to the economic saving in transportation.

Fair Value for Rate-Making.—The fair value for rate-making for railway properties should consider the whole territory served by the properties. Rates that are not sufficiently high to provide an adequate return for some lines within a territory would for other lines in that territory, which enjoy a less capital expenditure and less operating cost, produce more than what is usually considered as a reasonable return. The Committee's report considers the abnormal case of a property which must be a losing venture, but overlooks the abnormal case of a property which must be a sort of El Dorado.

Methods of Determining Physical Value.—The writer is in accord with the Committee in its conclusions that a valuation of an old property must necessarily be made after the cost-of-reproduction method; and that the identical plant is to be reproduced and not a substitute plant. These conclusions are particularly true with regard to railway properties. If the cost of substitute railways were proposed, the actual locations on the surface of the earth of the existing railway under investigation, the rates of grade, and curvature, etc., with all the theories of economic operation, would be a subject of inquiry, the end of which would be confusion. Furthermore, reproduction of the identical railway prohibits the process of inquiry from theoretically correcting errors of location and construction, which often become apparent after the completion of the railway. This is just to the owner. Errors of location and construction are an inevitable consequence of railway construction, and the burdens of such rest on the public. The

burden of an error of the owner in constructing a railway for which there is no economic necessity properly rests on the owner, except in those communities where the public through its laws and commissions, exercises the right of authorizing or denying the construction of any proposed railway. Mr. Willoughby.

Present, or Original, Conditions.—The conclusion of the Committee, that "the valuation should be based on the conditions existing at the time the various portions of the property were built, but on prices prevailing at or near the time of valuation", will result in confusion if applied to railway properties. Railway construction requires that there shall be the necessary materials of construction, machinery, teams, and tools of their several kinds, labor, skilled and unskilled, and transportation. The costs of construction work have a direct relation to all these items. When they are readily obtainable and cheap, the cost of the work is small; any scarcity of either causes the unit prices to advance. Some materials that were available (*e. g.,* cheap timber) when many of our railways were built, no longer exist; other materials (*e. g.,* cheap steel, explosives, and cements) were not then obtainable. As a result, many features of construction now called temporary were then adopted. It would be unjust to the public to capitalize in a present-day cost-of-reproduction valuation the trestles that were built long years ago and afterward filled, for a railway in a community where such methods are not at this day habitually practised as an economy. The convenience of transportation facilities to the railway subject to the valuation, affects enormously the unit costs of the work. The transportation conditions at the time of original construction, generally, were such that we cannot intelligently modify the unit costs attainable with modern excavating machinery to fit the original conditions, and we get unit prices and construction periods that are unreasonable. The difficulties recited by the Committee as making undesirable the use of present-day conditions are easier to overcome in railway valuations than the new difficulties that arise if we undertake to revive the original conditions. Take the Wachusett Reservoir. With present-day conditions, we would assume in case a highway or railway reaches one limit of the reservoir property, follows partly around the reservoir property and departs from it, that the highway or railway crossed the reservoir property, and was reconstructed on its present location, the cost-of-reproduction of which would be determined in accord with the principles and methods under discussion by the Society. An examination by borings would determine if the original soil were stripped from the reservoir site; and an examination of the depth of soil beyond the limits of the wetted area would determine the estimated depth of surface soil removed. The damages paid to the adjoining real estate, as incident to construction, would be estimated in accord with the experience of the inquiry as against all

Mr. Willoughby. real estate in the condition of improvement which that real estate exhibited at the date of the inquiry. If, at the date of the inquiry, there were buildings on all sides of the reservoir property, the assumption would lie that like buildings would extend over the whole reservoir site; if buildings were about a portion only of the reservoir property, the higher value of the improved real estate would attach to that proportion of the area of the reservoir which the improved portion bears to the whole perimeter of the reservoir site. Such a method would give to the owner the benefit of the decision of the Federal Supreme Court in the Minnesota rate case, and some part of the increment in the value of the property that is the just due of the owner. Take the illustration given of the New York Central Station. It is equity to assume that the old station was obsolete, and that its original cost had been returned to the owner through earnings during its life, and that a new station of the character provided was necessary for present use and the reasonable requirements of the future. To make a valuation after the cost-of-reproduction method, the assumption is made that all improvements on the station site are obliterated. If the inquiry is made, at the same time, of the cost-of-reproduction of the railways which are using the station, no injustice is done the owner, because there is no traffic to handle. If the inquiry is made as to the cost-of-reproduction of the station only, then the inquiry must make provision for the traffic, either by providing as a part of the cost of the new station other property for temporary use with temporary facilities, or by so augmenting the unit costs of the construction work as to permit the work to be undertaken with all the burdens of handling the traffic existing at the date of the inquiry. The burdens on the work, as a consequence of the proximity of other properties, rights and interests at the date of the inquiry, would rest in either case.

Application of Reproduction Method of Valuation.—The conclusion of the Committee, that "it is necessary to make a deduction from the values obtained * * * for the net accrued depreciation of the physical property due to age and other causes" does not necessarily follow for railway properties. Depreciation is merely an item of maintenance of some part of the plant deferred beyond the period of one year because the nature of that part does not permit the repair to be made within one year. If our accounting were in ten-year periods, instead of one-year periods, many of the items to be provided for as depreciation, and to be deducted from the value of the physical property under the Committee's conclusion, would be considered as items of maintenance, and, therefore, not deducted from the value of the physical property. The difficulty may be overcome by regarding depreciation as a deferred maintenance, to be taken into the annual accounting as a liability of the owner, but not to be deducted from the value of the physical property for rate-making purposes, or for

any other purpose, any more than any other liability is to be deducted from the value of the physical property, for illustration, the unpaid fuel bills. Being an item of indebtedness of the owner, the accrued depreciation at the end of any fiscal year should be accurately set out in the accounting so that the amount thereof would be a debit against the assets of the owner. The assets of the owner will include items in addition to the physical property of the plant, and it is unjust to the owner to reduce the value of his physical property by a liability necessarily incurred in the operation of the plant, and which his balance sheet shows to be within his ability to meet when the necessity arises, and then on such reduced valuation, to fix the rates to be charged for service, or to fix the amount of a purchase price. Mr. Willoughby.

Inventory.—The writer would add, to the Committee's illustrations, the familiar one of the depression of the underlying soil by the weight of railway embankments when constructed over the low-lying lands of the Coastal Plain in the South Atlantic and Gulf States, and on similar soils elsewhere. The amount of this subsidence is not determinable from any measurements, made after completion of the roadway, of the volume of the embankment, although the amount of such subsidence is often 20% and more of the total volume of the embankment.

Unit Prices.—Care must be taken in adapting present-day prices to the cost-of-reproduction method. The present-day prices most naturally chosen will be those current, on the work of the railway under inquiry, for additions and betterments, in connection with like prices on railways in the same general territory. Now, such prices are always based on facilities of transportation afforded by the railway to which the inquiry of cost-of-reproduction is directed, and on other concessions in the way of free transportation, etc. Proper additions should be made to such current prices to ascertain what would be the correct unit prices, were the railway and its branches assumed as being blotted out preparatory to its reproduction under present-day conditions.

Another feature having an enormous effect on unit prices is the demand made at any one time on the plants, organizations, and labor available for railway construction. In order that the assumptions necessary for a cost-of-reproduction may not become absurd, the amount of work proposed to be done, and the time limits set therefor, must be in reasonable accord with the available agencies for construction, all other business being assumed to be normal. This requirement can best be met for the larger systems by assuming that the reproduction work in any one State is undertaken separately from that of any other State, and that the construction agencies of all adjoining States are available. This assumption would fix as a maximum time limit the time necessary to reproduce that portion of the railway system under inquiry which exists in one State only. The suggestion of divi-

Mr. Willoughby. sion into States grows out of the probability that the results of the cost-of-reproduction valuation will be required for use in intra-state discussions as well as in interstate discussions.

The following assumptions may be made in endeavoring to adapt present-day prices in the older parts of the United States to the method of cost-of-reproduction:

1.—That the work will be done under all the present-day conditions; with the available contractors, and with labor of its present amount and efficiency, and with machinery of the several kinds now in daily use; and that the contract of the work will be burdened with all the prejudices, legal hostilities, and greeds held by the public against railway enterprises, which enable the contractors and the public to collect doubtful and unjust claims against the railway company.

2.—That the railway is to be built exactly as it exists at the time the estimate is made (even to repeating all the engineering errors that appear in the existing railway), and that the better railway, in the light of present-day needs, which might be built for the same money is not to be reproduced.

3.—That the railway to be reproduced must be considered as being entirely obliterated; the right of way re-afforested where any growing timber now touches either limit of the right of way, and as swamp land where swamp lands now border the right of way; as fields or meadows when such border the right of way, and as city or town lots for the length the railway exists within the limits of cities and towns. All cuts are to be considered as re-filled, and all embankments and structures of every kind as being entirely removed. These conditions are necessary to enable an estimate to be made of the amount of work to be done.

4.—That with the obliteration of the railway, the facilities of transportation afforded by the railway itself, or its branches no longer are available. There is a definite and determinable relation between the cost of the construction of the roadway, and structures of a railway, and the distances over which material, labor, and supplies must be transported by wagon haul.

5.—That all the facilities of transportation offered by rail lines other than the line to be reproduced, and by all water lines and highways, adjoining the line to be reproduced, are available at the current transportation rates and conditions.

6.—That when a system extending over many States is to be reproduced, it will be advisable to consider the work as being done in certain sections or zones, and to estimate the work of reproduction of each section or zone separately. This assumption is important because it is physically impracticable to construct the whole of an extensive system at one operation, without introducing fabulously high

construction costs, due to the lack of labor and appliances for such construction, when other business is normal in the United States. Mr. Willoughby.

7.—That the natural divisions into sections and zones will be by States. In any event, it will be necessary to tabulate the reproduction cost of work by States, and when it becomes necessary to use the reproduction cost in intra-state matters, any item of reproduction cost of the lines within the State that is affected by the assumption of the obliteration of lines without the State cannot be defended in a controversy over intra-state matters.

8.—That within any one State the main lines (both principal and secondary) should first be reproduced, and then the branch lines and spurs. Such an assumption will lessen the construction cost of the branch lines and spurs, because of the transportation facilities afforded by the main lines.

9.—That the time element is of great value, more than the mere calculation of interest on monies expended.

The purpose of these several assumptions is to make clear the conditions to be considered in determining the correct quantities and correct unit costs to be used in the calculations for cost-of-reproduction.

With particular reference to the lines composing the railway systems in the Atlantic Coastal Plain, Fig. 6, the varying character of the soils, native resources, transportation facilities, etc., forbid the adoption of a single unit price for the several items of railway construction, for the system as a whole, or even for all the lines in one State; for example, the dry soils of Florida are generally too soft and too light to permit the economical use of scraper outfits, and the wet soils are too swampy. This condition makes necessary the use of human labor. The dry soils of Southwest Georgia and South Alabama are generally indurated to such an extent as to be too hard for the economical use of scraper outfits, and at the same time, the transportation facilities are too meager to permit the bringing of steam excavating machinery and fuel therefor. This results in high unit costs, due to the use of scraper outfits and human labor on materials not economically adapted for such.

The unit costs for branch lines should generally be taken as less than for the main lines, to be in accord with the eighth assumption, and because the time element is not of such value as in the completion of main lines.

The fixing of unit costs can be arrived at by considering separately each strip of 50 to 100 miles of railway, and fixing unit prices for each strip, which (if there be any desire therefor) may be then averaged for the State as a whole.

As an example, consider the unit cost of grading required for the reproduction of the main line of the Atlantic Coast Line Railroad from the Virginia State line to Wilson, 62 miles. The single-track railway was

Mr. Willoughby. built more than 50 years ago. It has been constantly improved until it is now a double-track main line of the first class, carrying heavy and fast traffic. If the main line and branches are assumed to be obliterated there would remain the following rail and water transportation facilities: Atlantic Coast Line at Virginia State Line, Seaboard Air Line at Weldon, Norfolk-Southern at Wilson, Roanoke River at Weldon, Eastern Carolina from Tarboro south.

The present-day contract unit prices paid by the Coast Line in this territory are confined exclusively to construction of second tracks and other additions to the property. These unit prices were based on the privilege of free transportation of men and materials, etc., over an extensive mileage, and, to adapt them to the cost-of-reproduction method, with no free transportation, and none of any kind during the reproduction period, by that portion of the railway under inquiry, there should be added the following percentages:

1.—The value to the contractor of the free transportation over the Coast Line on employees, tools, machinery, construction material, and fuel for steam shovels, etc., in percentage of the contract price..10%

2.—The additional cost of construction on account of transporting men and animals, tools, machinery and supplies overland, from the nearest transportation facilities for distances in excess of an average wagon haul of 5 miles; to-wit: 16½ miles at 10% increase for each 5 miles $= 16.5 \times \frac{10}{5} = 33$ per cent. The 33%, however, applies only to the strip from Wilson to Weldon (85% of length Virginia State Line to Wilson), because there are transportation facilities within 5 miles of that portion of the main line between Virginia State line and Weldon, 33% of 85% =................28%

3.—The additional cost of construction due to the payment of higher wages to labor, necessary to induce labor to go to and remain on the line from Wilson to Weldon, remote from transportation facilities: 20% of 53% of length of line affected = 10.6 per cent. Inasmuch as the labor cost is only about one-half the total construction cost, the total increase in percentage becomes........... 5%

4.—The additional construction cost due to increases in prices resulting from the demands made on the labor and material markets of North Carolina were the 553 miles of principal and secondary main line of the Atlantic Coast Line to be reproduced as a single operation: 25% on 50% of total, in percentage..................12%

5.—The additional sum necessary to be procured by the contractor as profits to warrant him in handling the greater amount of capital: 5% of 55%.. 3%

Total increase..58%

Mr. Willoughby.

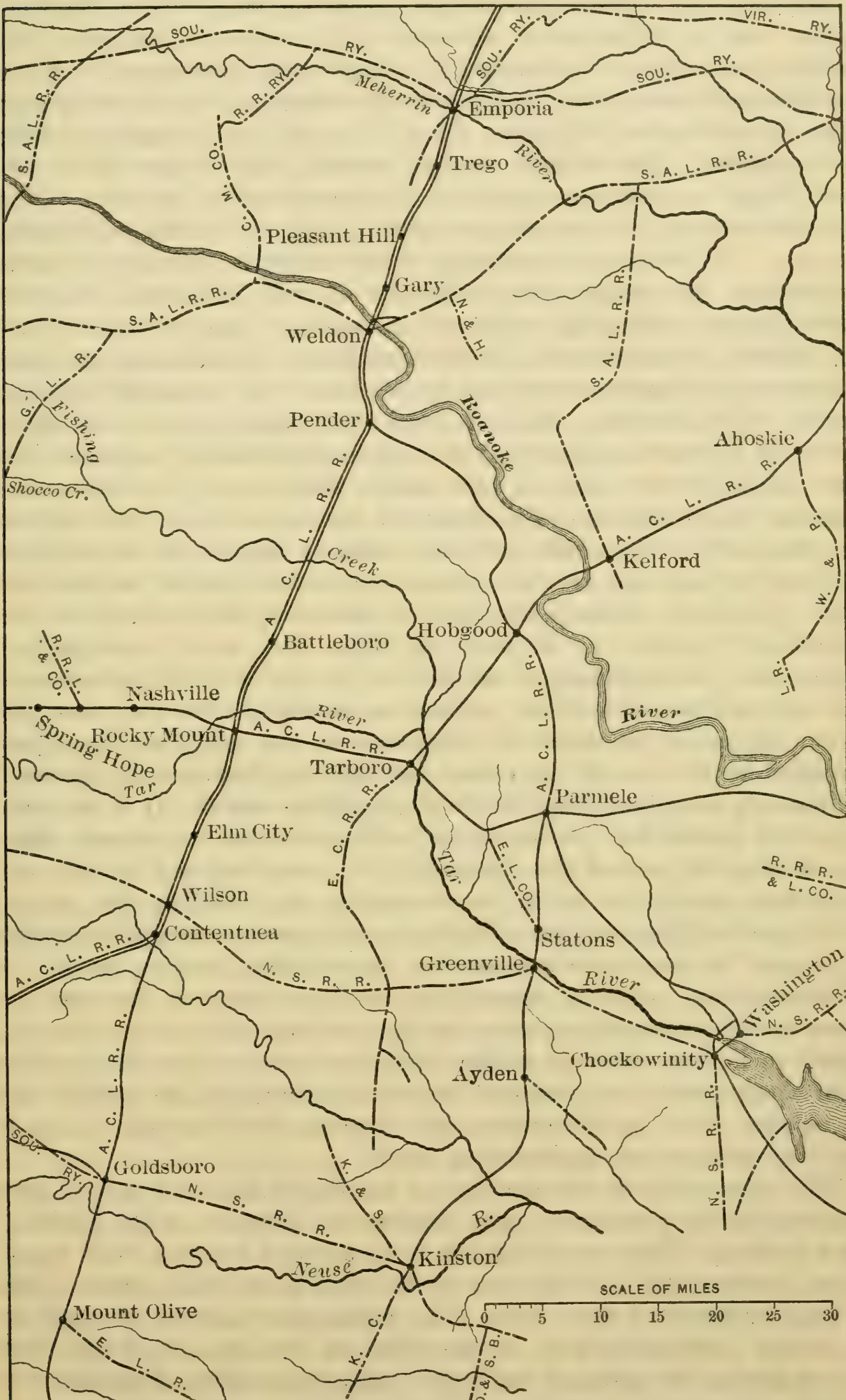

FIG. 6.

Mr. Willoughby.

Depreciation.—The writer cannot accept the statement of the Committee, that "a depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment." Exclusive of consideration of rolling stock, depreciation in the physical properties of railways accumulates quickly on rail, fastenings, and other track material, ties, ballast, wooden trestles, and bridge floors; and more slowly on buildings and the superstructure of bridges. Other than obsolescence, it accumulates very slowly on bridge and culvert masonry. The current accounting of our railways charges to annual maintenance the entire replacement cost of rail, fastenings, and other track material (unless the weight be increased at time of replacement) ties, ballast, wooden trestles, and bridge floors. Buildings and bridge superstructure (unless destroyed by accident) are generally replaced because of obsolescence. In such cases the original cost or cost of replacement in kind (less salvage) is charged to operating expenses. The same rule applies to masonry and similar structures. The current accounting of our railways takes from the annual revenues derived from the rates paid by the public sufficient sums to replace the depreciation maturing in any one year as operating expenses, before paying dividends. After the railway has been in operation long enough for depreciation to mature on the principal items, the annual operating expenses care for depreciation as well as for the annual maintenance. The only injustice which the public can assert is that the rates paid during the period in which the depreciation was maturing have been converted to the use of the owner without an allowance for interest, and possibly have been used for dividends to the owner. It is the opinion of the writer that justice to the public requires an interest allowance during the period the depreciation is maturing and to that end the railway should set up in its accounting as a liability the amount of the accrued depreciation. Maintenance expenses would thereafter be charged for any year against the depreciation fund so set up for all depreciation accruing during the previous years on the item replaced and to operating expenses for the year the depreciation accruing during such year. Justice to the public does not require the creation of an actual trust fund invested in outside securities, but merely that the balance sheet of the owner shall disclose his financial ability to meet the demands on depreciation fund.

This view prohibits the adoption of the Equal-Annual-Payment plan suggested by the Committee, and requires the adoption of the Sinking-Fund Method. The rate of interest to be adopted for such fund would be low, because the security back of the fund is the whole value of the physical property of the owner. The public can demand, through its regulating authorities, that in the event of the amount of the fund ever exceeding the assets of the owner, other than value of the physical property, then the amount of the fund to the extent of such excess shall

be deducted from the value of the physical property with the resulting lower service rates. Mr. Willoughby

Adaptation and Solidification.—Wherever the growth of grass has the effect of sod to prevent washing from the slopes, the roadway has been improved, and the cost to the owner of procuring this benefit should be added to the value of the physical property. If the roadway be in a section of the country where sod grows naturally, as in Central Kentucky, the total cost of procuring the sod will be the cost of repairing the wash from the slopes until the sod forms.

S. S. Roberts, M. Am. Soc. C. E. (by letter).—In any business, private or corporate, for private or public service, the expectancy giving rise to its creation is that there will be a return which will cover the cost of organization, construction, running repairs, renewals, and operation, an allowance for risk, and interest on the capital invested, which is the basic principle laid down by the Committee as follows: "A public utility property is entitled to have the rates made high enough" so that there shall be received, "not only a fair return for the use of the capital invested in such property, but shall also have the capital invested in it paid back." The consensus of opinion indicates this to be certainly the minimum return to justify any undertaking. Mr. Roberts.

The report deals entirely with public service utilities, and especially railroad property. Such properties are in their nature perpetual. The paying back of the capital consists in furnishing that needed for renewing parts of the plant as they wear out. Just as in any business, in sale of actual commodities, the sales supply means for purchase of new stock. In case of a railroad, there is this difference: the transaction is the sale of the use of the stock, and not the stock itself. The buyer, therefore, is not concerned with the age and condition of the stock so long as it is capable of supplying the use or service paid for.

When a business for the sale of commodities ceases to supply stock, it ceases to exist. When a railroad ceases to supply service, it ceases to exist as a public service property, as far as its owners are concerned, and becomes a business in sale of commodities through forced change of owners. The age and condition of the parts of the property are of essence only in involuntary or voluntary transfer. The obligation to furnish service is assumed by the new owners, and the errors of omission or commission of the past are taken care of in the sale price.

As long as service is furnished, for which only a fair return is paid, the public should consider it immaterial how the service company handles its funds, and makes renewals. In other words, whether the service company maintains a sinking or any other kind of fund, has nothing to do with the public's side of the question.

The preponderance of opinion and discussion on valuation for rate-making is clearly in support of the method of "Cost-of-Reproduction-

Mr. Roberts. New" without depreciation. This places the old and new companies on a more nearly equal basis, and allows the older companies the advantage of the appreciation of their property, which should be their just due in reward for their foresight and risk in undertaking pioneer work.

The writer feels compelled to disagree with the Committee's conclusion as to appreciation. The growth and development of the community and of the railroad serving it are identical. The fact that the property of the community appreciates is due mainly to its advantage of location, which is due to the operation of the railroad to and through it. Their right to enjoy what they have is not questioned. At least, it is not in this consideration of rate-making. The advantage of the railroad is also one of location, whether determined through foresight or chance, which places it in a community capable of development and appreciation through transportation service. Advantage of the appreciation of the community property may be taken only through sale, mortgage, or increased rental. The same is true of railroad property. The appreciation is latent. Depreciation is an operating expense active and dynamic, a renewal of property actually given to the community together with the service rendered; it should be paid for as such, and should not be used to offset appreciation.

Some have argued, in support of the Committee's view, that the right of "eminent domain" is a special privilege to the railroad for which the public should take toll. It is true that very few railroads could be built without this right, but do they not pay full value for all property they acquire under it; and should they not enjoy all the advantages and rights of other property holders who have paid full value for what they possess? Is not the power of eminent domain, as a matter of fact, a question of public policy, so that the individual may not gain at the expense of the many, and is it not created for the public good?

To balance depreciation against appreciation seems to be taking toll twice from the railroad company. Unless the whole system of land values were changed, affecting alike the community and the railroad, is it just to take that from the railroad which is freely allowed to the remainder of the community as its just award?

Fair agreement seems to obtain on all points for which return should be permitted. The most mooted consideration remaining is the manner of providing adequate and, at the same time, equitable means for renewals, so that the return for this purpose may be spread over the period of usefulness of the parts so as to give a total return which shall be reasonably uniform and result in steady, not fluctuating, rates.

It is too trite to mention that he who deals in the future, deals with uncertainty; yet the fact remains that our best decision in regard to provision for renewal funds is an estimate. The one who must bear

the result certainly should be given the advantage of all reasonable doubts. Mr. Roberts.

Alexander C. Humphreys, M. Am. Soc. C. E., in his paper, "Depreciation: Estimated and Actual," shows the error of considering the average life of a plant as a whole in making provision for depreciation, or, as the writer prefers to express it, renewals. Mr. Humphreys emphasizes the necessity of considering separately each group of parts having the same period of usefulness. He and others give warning that, in order to hope for results sufficiently near to the probable actual performance of any part to warrant consideration, careful account must be taken of the past history of the part and of its present condition, as determined by actual examination.

Considering that the period of usefulness of a part has been determined as accurately as human knowledge and foresight will permit, now comes the problem of providing a means to secure its perpetuation.

The methods of computation form the academic considerations of the problem; the equity lies in the adoption of that method which gives just results under all conditions.

There is agreement between all writers that the Sinking-Fund Method of computation, considering return on the "Cost-of-Reproduction-New" without depreciation, is correct in ultimate result under all assumptions of rate of interest for return on invested capital and for computation of the annual increment for depreciation or renewal. It makes no difference whether the rate of interest for return on invested capital is the same or different from that used in computing the annual sinking fund or renewal increment.

It is generally conceded, too, that such a basis of computation represents a minimum permissible return, because the annual increment, must, on its receipt, be at once put out at compound interest in order to equal the necessary accumulation during the period of usefulness of the part.

Where the rate of interest allowed for return on invested capital and for computation of the annual depreciation or renewal increment is the same, the ultimate result of the Sinking-Fund Method and of the Equal-Annual-Payment Method is identical. This is shown in Table 16.

C. E. Grunsky, M. Am. Soc. C. E., in his paper, "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates",* in "Table 8, Method 1", and "Table 8, Method 3", and also in his Fig. 3, and in his explanation of the different methods, confirms Table 16.

It is evident that where the rate of interest is the same, there is a distinction, but not a difference, in these two methods, and any difference in distribution of the total annual return is imaginary.

* *Transactions*, Am. Soc. C. E., Vol. LXXV.

Mr. Roberts.

TABLE 16.—PERIOD OF USEFULNESS, 20 YEARS; RATE OF INTEREST, 5 PER CENT.

Age, in years.	Sinking-Fund Method.			Equal-Annual-Payment Method.			Ultimate or total return, either method.
	Capital invested.	Annual renewal increment.	Annual return on capital invested.	Depreciated value of capital invested.	Annual depreciation or renewal increment.	Return on depreciated capital.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	$100.00	....	....	$100.00	....	....	....
1	100.00	$3.02	$5.00	96.98	$3.02	$5.00	$8.02
2	100.00	3.02	5.00	93.80	3.18	4.84	8.02
3	100.00	3.02	5.00	90.47	3.33	4.69	8.02
4	100.00	3.02	5.00	86.97	3.50	4.52	8.02
5	100.00	3.02	5.00	83.29	3.68	4.34	8.02
6	100.00	3.02	5.00	79.43	3.86	4.16	8.02
7	100.00	3.02	5.00	75.38	4.05	3.97	8.02
8	100.00	3.02	5.00	71.12	4.26	3.76	8.02
9	100.00	3.02	5.00	66.65	4.47	3.55	8.02
10	100.00	3.02	5.00	61.96	4.69	3.33	8.02
11	100.00	3.02	5.00	57.03	4.93	3.09	8.02
12	100.00	3.02	5.00	51.86	5.17	2.85	8.02
13	100.00	3.02	5.00	46.43	5.43	2.59	8.02
14	100.00	3.02	5.00	40.73	5.70	2.32	8.02
15	100.00	3.02	5.00	34.74	5.99	2.03	8.02
16	100.00	3.02	5.00	28.45	6.29	1.73	8.02
17	100.00	3.02	5.00	21.85	6.60	1.42	8.02
18	100.00	3.02	5.00	14.92	6.93	1.09	8.02
19	100.00	3.02	5.00	7.64	7.28	0.74	8.02
20	100.00	3.02	5.00	0.00	7.64	0.38	8.02

The Equal-Annual-Payment Method, as compared with the Sinking-Fund Method, the basic rate of interest being the same, is simply the shifting of a part of the contents of one of two pockets to the other, without changing the total contents of the pockets taken together.

The foregoing comparison ceases to be true when, for any reason, the rate of return on the capital invested is different from that used in computing the depreciation or renewal increment:

(1) When the rate of return on the invested capital is greater than that allowed in computing the renewal increment, then the Equal-Annual-Payment Method ceases to be an Equal-Annual-Payment Method in any sense of the word, and will work a continually increasing injustice on the public service company after the end of the first year until the end of the period of usefulness of the part under consideration. This is shown by Table 17.

(2) When the rate of return on the invested capital is less than that allowed in computing the renewal increment, then the Equal-Annual-Payment Method again ceases to be an equal-payment method in any sense of the word, and will work a continually increasing injustice on the rate-payers after the end of the first year until the end of the period of usefulness of the part under consideration.

TABLE 17.—PERIOD OF USEFULNESS, 20 YEARS; RATE OF RETURN ON CAPITAL INVESTED, 7%; RENEWAL RATE, 5 PER CENT. Mr. Roberts.

Age, in years.	Sinking-Fund Method.				Equal-Annual-Payment Method.			
	Capital invested.	Annual renewal increment.	Annual return on capital invested.	Ultimate or total return.	Depreciated value of capital invested.	Annual depreciation or renewal increment.	Return on depreciated capital.	Ultimate or total return.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	$100.00	....	....	....	$100.00	....	....	....
1	100.00	$3.02	$7.00	$10.02	96.98	$3.02	$7.00	$10.02
2	100.00	3.02	7.00	10.02	93.80	3.18	6.79	9.97
3	100.00	3.02	7.00	10.02	90.47	3.33	6.57	9.90
4	100.00	3.02	7.00	10.02	86.97	3.50	6.33	9.93
5	100.00	3.02	7.00	10.02	83.29	3.68	6.08	9.76
6	100.00	3.02	7.00	10.02	79.43	3.86	5.83	9.69
7	100.00	3.02	7.00	10.02	75.38	4.05	5.56	9.61
8	100.00	3.02	7.00	10.02	71.12	4.26	5.27	9.53
9	100·00	3.02	7.00	10.02	66.65	4.47	4.98	9.45
10	100.00	3.02	7.00	10.02	61.96	4.69	4.67	9.36
11	100.00	3.02	7.00	10.02	57.03	4.93	4.33	9.26
12	100.00	3.02	7.00	10.02	51.86	5.17	3.99	9.16
13	100.00	3.02	7.00	10.02	46.43	5.43	3.63	9.06
14	100.00	3.02	7.00	10.02	40.73	5.70	3.25	8.95
15	100.00	3.02	7.00	10.02	34.74	5.99	2.85	8.84
16	100.00	3.02	7.00	10.02	28.45	6.29	2.43	8.72
17	100.00	3.02	7.00	10.02	21.85	6.60	1.99	8.59
18	100.00	3.02	7.00	10.02	14.92	6.93	1.53	8.46
19	100.00	3.02	7.00	10.02	7.64	7.28	1.04	8.32
20	100.00	3.02	7.00	10.02	0.00	7.64	0.53	8.17

NOTE.—In Tables 16 and 17, the computations of the Committee are used.

A table will not be given to illustrate this. Such a premise can hardly be imagined to obtain in practice. It is sufficient to state that, considering a rate of return on capital at 5%, and computing the replacement increment at 7% for a period of usefulness of 20 years, the ultimate or total return at the end of the first and the twentieth years will be:

	Sinking-Fund Method.	Equal-Annual-Payment Method.
1st year..................	$7.44	$7.44
20th "	7.44	9.26

The Sinking-Fund Method and the Equal-Annual-Payment Method are thoroughly discussed by Mr. Grunsky in the paper, previously referred to, as "Method 1" and "Method 3", respectively, and their relative advantages and disadvantages are described by him as follows:

"Method No. 1 [Sinking-Fund Method] is always applicable when it can be shown that earnings have been adequate in the past, no matter whether the property is a single item, or is composed of many items; whether the expectancy is long or short; whether the expectancy is uniform for all parts of the property or not; whether the plant is of mature age or not; whether the property has attained full growth

Mr. Roberts.

or whether it is still growing at a uniform rate, or otherwise. It is simple of application, and does not involve determination of the present condition of the property, provided that it is maintained in proper condition to render adequate service. It furnishes all the information necessary for intelligent action in fixing rates, because when it is known what the net earnings, above operating expenses, must be to yield the return which money should earn in ordinary safe investments, then an arbitrary addition can be made, to compensate the owner for management and risk of loss.

"In contrast with these advantages, Method No. 3 [Equal-Annual-Payment Method], under which value as ordinarily understood must be determined by deducting depreciation from the investment, requires a special determination of value for each item of which the property is composed, and a new determination every year for every item, or in special cases, for every group of items of the same expectancy. Each item has a new value each year and a remaining life which grows continually shorter. Amortization, therefore, must be estimated on a new basis each year. The judgment of the expert is called into play to determine the condition and probable remaining life of the several parts of the property, and after the complex calculation is made, if the same basic rate of interest is used throughout, the result should agree absolutely with the simpler Method No. 1."

No greater part of the investment is returned to the investor by the Equal-Annual-Payment Method than by the Sinking-Fund Method. The Equal-Annual-Payment Method is more difficult to compute, and gives correct results only under the condition of equal basic rates of interest.

If Table 16 is correct, then in Table 6,* of the Committee's report it is evident that part of the return on the capital invested is shown reinvested in additional plant as "earnings for depreciation". Also, at the end of the first year, after the depreciation or replacement increment of $7 238 is reinvested in additional plant, the plant is furnishing all the service any $107 238 plant could possibly furnish. The original $100 000 investment and the $7 238 investment, as far as the plant and service are concerned, and as far as ultimate return is concerned, should be treated as entirely separate and distinct investments, each of which enjoys in every respect the same and all rights and privileges accorded to capital to earn.

The illustrations of notes and their payment used by the Committee do not appear to be in all cases *apropos,* for the reason that the perpetual or continuous nature of the property is not considered. That is, that something in railroad properties, termed "Latent Depreciation", by Richard J. McCarty, M. Am. Soc. C. E., in his paper, "Depreciation of Railroad Property", is not taken into account.

The adoption of the term "depreciation fund" seems, in a way, extremely unfortunate, and to it may be attributed much of the mis-

* "Addition to the Progress Report of the Special Committee."

understanding of the purpose for which it is created; especially in the minds of those not thoroughly familiar with the whole subject. Although, of course, this notation arises from the fact that the basis of computation of the fund is wear and tear, and final exhaustion, still the fund is not for depreciation, but for continuation and perpetuation, and is, it seems, more happily expressed as a "Renewal Fund", which certainly is its purpose. The rate-payers get the whole of the plant, along with their service. The "Renewal Increment" is proportional to the rate at which the plant is used up by them. The "Renewal Fund" is something due the service company, not something to be taken from it. Mr. Roberts.

The writer believes the "Sinking-Fund" or "Renewal-Fund" Method to be generally more equitable than the proposed "Equal-Annual-Payment" Method.

Considering that it requires all the "Renewal Increments" increased by all their interest accumulation to equal the "Renewal Fund"; considering the uncertainty of forecasting future performance; considering the advance of the art which frequently requires the renewal of parts before their expected period of usefulness can be enjoyed; considering the mature age of the majority of railroad properties, which have reached that stage where renewals are uniform and constant; that is, considering the "risk", liberality should be used in dealing with the allowance for and make up of the "Renewal Fund".

C. P. Howard,* M. Am. Soc. C. E. (by letter).—The "Sinking-Fund" Method provides: Mr. Howard.

(1).—Equal annual payments, which at compound interest will amount to the full value of the plant at the end of its life.

(2).—Return on capital computed for the full value of property.

The owner then receives a return (say, 7%), on the full value of his investment throughout its life; and the sinking-fund increments put out at compound interest (say, 5%), will return him the full value of the plant at the end of its life. If these increments, computed on the basis of 5% interest, were by good management invested at 7%, the owner would have a surplus of more than enough to renew his plant.

The "Equal-Annual-Payment" Method provides for the repayment of the invested capital. Mr. Roberts points out that this gives a considerably less return to the investor than the Sinking-Fund Method, when depreciation allowances are calculated at a less rate of interest than return on capital.

The reason for this is obvious. If the owner could re-invest immediately each depreciation allowance in the plant, or in other business, which would earn 7%, without any loss of time, or development period, he would at all times have 100% capital invested with annual returns

* Previous discussion by Mr. Howard appeared in *Proceedings* for February, 1914, on p. 387.

Mr. Howard. of 7%, as indicated in Table 6, of the Progress Report. Of course, he could do better than this with the larger payments received under the Sinking-Fund Method; but the very reason for computing sinking-fund increments at a lower rate of interest is the supposition that capital must be kept on hand and available for more or less irregular expenditures, as they occur. It must be ready cash. If it is re-invested, the company will have to go to the expense of borrowing on short-term notes or some other method of securing ready cash.

Referring again to Table 6, and what it implies: Is it reasonable to suppose that the company can always immediately re-invest every dollar in improvements and extensions? Except to a limited extent, it is evident that improvements and extensions cannot be planned and financed in such manner; and, if they could, it would be idle to expect full earnings from the start. It is possible, of course, that a part of the fund might be re-invested from time to time at 7%, the rest remaining on deposit at a lower rate of interest, say 3 per cent. The average interest return on the sinking fund would then be somewhere between—say, 5 per cent.

The matter could be handled easily enough under the Sinking-Fund Method, whatever disposition is made of the fund. The only thing to decide is what is the reasonable average rate of interest at which the sinking fund may be expected to accumulate when invested under ordinary conditions with reasonably good judgment.

Mr. Mortimer. James D. Mortimer, Esq. (by letter).—Judging by the title, the Committee has confined its preliminary report to valuation for the purpose of rate-making. There are other purposes for which a valuation may be made, such as purchase and sale, taxation, municipal or governmental acquisition, etc., which would introduce other elements of value.

As the report is preliminary and the discussion now called for partakes of a like nature, the writer's remarks are purposely brief, and designed to sketch the line of investigation of statistical and economic principles now being conducted by several public utilities and the Bureau of Fare Research of the American Electric Railway Association.

1.—The process of assembling the elements of value is very much simplified by concentrating the attention on the property of the utility, as distinguished from that of the corporation owning it. The utility is the entity possessing the franchise or permit to occupy the streets, the physical property used or useful in the performance of the public service, and the other assets necessary to the operation of the complete plant. This conception is much simpler than that of principal and agent, which some have recently endeavored to introduce for the purpose of defining the relation between the State, the public, and the public utility corporation.

2.—The determination of the valuation for the purpose of rate-making consists of assembling the elements of value or assets of the utility. These assets consist not only of the physical property used or useful, but also all other assets, such as accounts receivable, which the utility may have owing to it by the corporation. Many of the reserve liabilities of the corporation, such as depreciation reserve, fire insurance reserve, injuries and damages reserve, are, in the proper accounting conception, assets of the utility and entitled to consideration in determining its value. Mr. Mortimer.

3.—In dealing with the subject of depreciation it is of importance that the various conceptions be recognized and distinguished, *viz.:*

(*a*) Rate at which depreciation is accruing at the moment or during the current year;

(*b*) Rate at which depreciation has accrued during the previous life of the unit of plant;

(*c*) Rate at which depreciation will accrue during the remaining life of the unit of plant;

(*d*) Annual reserve required to finance replacements as they occur from time to time;

(*e*) Accumulated depreciation measured by loss in value or utility;

(*f*) Accumulated depreciation reserve necessary to finance replacements as they occur from time to time.

No satisfactory definition of accrued depreciation or depreciated value has yet been developed which does not bear some relation to the estimated amount of money which would have accumulated under some plan of annual payments commencing at the time the unit of plant was placed in operation. Whether the problem of measuring accrued depreciation scientifically, by either inspection or statistics, will be separated from that of financing replacements, cannot now be predicted with any certainty.

4.—The actual future life of any particular unit of plant ascertained by inspection is indeterminate. Only the boundary conditions are questions of fact; the unit may be either new or about to be abandoned. With respect to all intermediate conditions, the facts are indeterminate; abandonment may occur to-morrow or 10 years hence.

5.—Experience and general observation show that there are usually two concurrent forces acting to limit life, that is, to require replacement or abandonment. These two forces are probably present in different relative proportions in each class of equipment, and it may be that in many classes either one or the other may be almost entirely absent. These two forces are defined as follows:

(*A*) The force causing destruction, remaining constant throughout the life of the unit, acting equally on new and old, and covering destruction arising from chance, such as accident,

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governmental requirement, inadequacy, improper design, and engineering errors in design, selection, and construction.

(*B*) The force causing destruction, increasing directly with age, acting on the old in greater proportion than on the new, and covering destruction arising from wearing out or other causes which vary directly with age, such as obsolescence and natural decay.

The relative proportion in which these two forces are present in any class of equipment cannot be determined until after the causes of depreciation have been quantitively measured in the light of a proper gathering and assembling of actual data.

If L = proportion of original number living at age, t; then,

$L = e^{-At}$, where only the force, A, is present;

$L = e^{\frac{-Bt2}{2}}$, where only the force, B, is present; and,

$L = e^{-t\left(A + \frac{Bt}{2}\right)}$, where both forces, A and B, are present.

Where e is the base of the natural system of logarithms.

6.—The problems involved in the determination of the proper amount to be included in operating expenses, to insure the replacement of the various plant units, are similar to those involved in the calculation of life insurance premiums. The amount, per dollar invested in a particular plant unit, required for such property replacement insurance depends on the age of the unit, the mortality table of a group of such units, the reserve already accumulated, the rate of interest allowed on accumulations in the reserve, and the evident state of preservation of the unit. The accumulations in reserve may be drawn on at any time, depending on the contingencies requiring the abandonment of the unit of plant. As the maturity dates of the loans making up the depreciation reserve of the utility are indeterminate, and may be one day or several years, the loan is entitled to a rate of interest comparable with that allowed on daily bank balances or the rate of interest assumed in calculating life insurance premiums, say, 3 per cent. This interest rate should be distinguished from that assumed for depreciation calculations, where an average life is estimated and the deficiencies in interest and reserve accumulations arising from early mortality are assumed to be offset by the greater interest accumulations during years beyond the assumed mean life. If the replacements required in a typical complete plant be set down in detail during a period of time running beyond the average life of the plant, it is found that the allowable interest rate resulting from interest accumulations is very low, approximating 2% in the case of some plants having an estimated average life in excess of 30 years.

7.—The rate of interest allowed on depreciation reserve accumulations is a very different quantity from the rate of return to be allowed on permanent investment in plant; the former may be withdrawn in large part at any time, but the investment in operating plant must remain until worn out or otherwise abandoned. It is also important at this place in the discussion to recognize the fact that public utilities as viewed in law are presumed to operate in perpetuity, and the owners of a utility cannot withdraw the capital invested in any plant unit when the depreciation reserve has returned an amount equivalent to the original investment. These owners, by virtue of the public functions assumed on behalf of the utility, are required to continue the plant in operation and make the needed replacement. In view of the obligations assumed by the utility, it is unreasonable to assume that accumulations in the depreciation reserve, created, as they may be, from payments by the users of the service, are in the nature of repayment to the owners of the utility of their capital investment. Only in the case where such depreciation reserve accumulations were distributed in the form of interest and dividends on securities of the corporation can it be said that there has been a withdrawal of the invested capital. Mr. Mortimer.

8.—From the nature of the property making up a public utility, the problems of valuation and rate-making should be the same, whether the utility be owned by a municipality, the State, an individual, a partnership, association, or corporation. Regulation has primarily to do with the functions of the utility. In the broad sense, the law has no interest in the nature of ownership of the utility, except to the extent that it prescribes and limits the powers of the particular entity in which such ownership may be vested. If these statements be true, it follows that capitalization, dividend record, and ratio between outstanding bonds and outstanding share capital, have no direct relation to the regulation of rates of a public utility owned by a corporation. Likewise, the rate-payers have no interest in stock dividends or book surplus appearing on the books of the corporation.

9.—These considerations permit one to draw the distinction existing between the "going cost" of the utility and the "going value" of the corporation, shown in the latter case by the excess of market value of securities over the cash cost of property acquired thereby.

10.—Specific comments might be made on various parts of the Committee's report, but it is doubtful whether space and time available for the present discussion will permit. Summarized, the principal defects in the Progress Report of the Committee are as follows:

(*a*) An erroneous conception of the function of depreciation allowances is adopted. Depreciation allowances are not repayment of capital unless actually disbursed as interest

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and dividends, and then only when and to the extent that the utility has earned a reasonable return.

(*b*) Depreciation reserve credits appearing on the books of the corporation are among the working assets of the utility, and hence additive to the reproduction cost for the purpose of determining valuation for rate-making; and, if any utility and its corporation have reserved 100% of the estimated accrued depreciation reserve, there results a sum equal to the reproduction cost new.

(*c*) Permanent investments in operating plant, of depreciation reserve accumulations, are entitled to the same rate of return as are investments of the proceeds of bonds and share capital.

(*d*) The very large probable errors contained in all present-day estimates of depreciation allowances, arising from lack of data covering the useful lives in service of various units of plant, do not justify extended discussion as to the relative merits and equity of the "straight-line", "sinking-fund", and "equal-annual-payment" methods. When the principles of actuarial science are properly applied to the problems of depreciation, much minute discussion of unimportant arithmetical methods will be avoided and some definite progress will be achieved.

(*e*) Substantial revision in the economic theories and mathematical principles of the Committee is required before its report can receive general adoption by engineers, public utility operators, regulating commissions, and the Courts.

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CHARLES S. CHURCHILL, M. AM. SOC. C. E. (by letter).—After carefully studying the report of the Special Committee, and after reading several published discussions thereon and listening to an explanation by the Chairman, Mr. Stearns, followed by several verbal discussions, the writer is greatly disturbed by the differences in interpretation of its meaning and the consequent dangers that may result in case it is accepted as the views of the Society on this particular subject.

The writer is particularly impressed with the thought that the questions raised therein are as much legal as engineering in character, or more so, and while not criticizing the Committee for taking a different view, he nevertheless felt himself not altogether competent to pass judgment on these legal questions. He, therefore, submitted the report to a legal friend for his opinion. In much of what follows the writer is indebted for suggestions to this friend, who, though he has no desire to take part in the discussion, has no objection to this use of his opinions.

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In its opening statement the report advises that the Committee preparing it was appointed in September, 1911 (which was prior to the enactment of the Federal Valuation Law), and that it was "to formulate principles and methods for the valuation of railroad property and other public utilities", but that it has submitted "a report covering the subject of Valuation for the Purpose of Rate-Making". Thus, what the Committee was to do, according to its own statement, and what it has done, as appears from its report, do not agree.

Subsequent to the appointment of the Committee, Congress enacted a Valuation Law applicable to "all property owned or used by every common carrier" subject to the provisions of the "Act to Regulate Commerce", which law became effective by the approval of the President on March 1st, 1913. The report of the Committee is made as of December 1st, 1913, for submission to the Society at its Annual Meeting on January 21st, 1914, and yet this Federal Law, which is absolutely controlling in the valuation of railroad properties, and had been in force for 9 months prior to the date of the report, is not referred to in that report, nor was the report prepared with any reference thereto, as is shown by its contents.

The purpose of the Federal Valuation Law is to obtain the value of the various railroad properties of the United States. This law in its opening statement provides that the Commission "shall, as hereinafter provided, investigate, ascertain, and report the value of all the property owned or used by every common carrier subject to the provisions of this Act". Further, it provides that "such investigation", by the Commission, "shall show the value of the property of every common carrier as a whole, and separately the value of its property in each of the several States and Territories and the District of Columbia, classified and in detail as herein required"; it provides that "All final valuations by the Commission and the classification thereof shall be published and shall be *prima facie* evidence of the value of the property in all proceedings under the Act to Regulate Commerce, as of the date of the fixing thereof and in all judicial proceedings, etc."; it provides that the Commission, in addition to investigating, ascertaining, and reporting "original cost to date", "cost of reproduction new", and "cost of reproduction less depreciation", shall, in like manner, "ascertain and report separately other values and elements of value, if any, of the property of such common carriers".

Judge Prouty, Director of Valuation under this law, in a speech on February 11th, 1914, before the United States Chamber of Commerce, said "Up to the present time the holding of the Supreme Court of the United States is, that the cost of reproduction new, or cost of reproduction less depreciation, are only factors entering into the final question of value. Many other things have been enumerated by that Court as bearing upon the value of the property". Again, in the same

Mr. Churchill. speech, he said "I simply call your attention to the fact that the Commission is required, not merely to ascertain the cost of reproduction, but to state the value of the property, and that in attempting to do so many delicate and difficult questions may be encountered".

The law, thus ascertained, clearly directs that "the value" of the property shall be ascertained. The purpose of this value, whether for the regulation of the issuance of securities, for the protection of investors, as suggested by Judge Prouty, for making rates or for testing or limiting them for taxation, or for some other purposes, is not stated. The valuation may serve many purposes, but whatever the purpose, it is "the value" of the property that must be considered. The law requires "the value" to be reported. Thus it reads and thus has Judge Prouty publicly construed it. No reason, therefore, exists and no purpose is now to be served, when the great work of ascertaining "the value" of "all the property" of every common carrier subject to the law is going on, for discussion, consideration, or conclusions on a particular kind of value, namely, a value for rate-making purposes, because such special values as this are not now under consideration by the Government and are not involved under the Valuation Law.

In the Summary (page 70 of the report), Paragraph 1 reads:

"This report is limited to the discussion of valuation for the purpose of rate-making. Valuations for other purposes differ in some respects, for legal and other reasons".

Also, on page 1 of the report, the Committee says:

"Valuations of the property of public service corporations may be made for a variety of purposes, the most important of which are the furnishing of a basis for the regulation of rates, and for taxation, capitalization and purchase".

It is manifest from this language and from a reading of the entire report that the Committee has been more concerned with a certain "purpose" of valuation, namely, rate-making, than with valuation itself. The Committee proposes methods for arriving, not at "the value" of the property, but at a sum or an amount of money, which, while it will not be the value of the property, will be the value on which the Committee recommends the carrier should be permitted to earn.

The Committee is not happy in the language it uses, as a justification for its course, namely:

"It is sometimes held that valuations for all these purposes [regulation of rates, taxation, capitalization and purchase] should give identical results. This view cannot prevail if the laws of the different States and certain decisions of the Courts are complied with. For instance, there are many State laws indicating how the property of public service corporations shall be valued for taxation and these laws differ radically in their requirements. There are other reasons why the valuation for different purposes should differ, but this feature will not be

discussed at present as this report will, as stated, relate to valuation for the purpose of rate-making". Mr. Churchill.

The omission of these "other reasons" and of any reason whatever for the statement that value for capitalization, for purchase, and for rate-making, is not the same, detracts greatly from the report. The mere fact that one State may tax public service corporations on income, another State by license, and another by *ad valorem*; or that one State using the *ad valorem* method may assess the property at one percentage of its value and another State, using the same method of taxation, may tax at a different percentage of value, illustrates nothing as to value. One State prefers one method of taxation and another State a different method. The fact that different States use different methods of taxation is not even suggestive that the measure of "the value" of the property varies in the different States.

To assume, as the Committee evidently has done, that the American Society of Civil Engineers has instructed it to report on the valuation of railroads "for rate-making purposes", or to assume that the Society will sanction the report though unauthorized, is, in each case, scarcely authorized. In the first case the directions to the Committee are in opposition to the assumption, and, in the second case, the long-established reputation of the Society for the broad treatment of all subjects coming before it equally opposes the presumption there indulged. When has any State ever made a tariff of rates or an individual rate for any railroad based on the value of the railroad's property? What State has ever proposed to do so? How would it be possible for a State or for the Federal Government to do so? A railroad may invoke the value of its property, as has been done, to show that an entire body of rates is confiscatory. In such case, however, it is not some peculiar kind of special value that is invoked, but "the value" of the property. The Constitution does not recognize fantastic differences over supposed different kinds of values, but it protects "the value" of all property, whether that of a common carrier or of some one else. It may be desirable for the Government—State or Federal—to be possessed of official information as to "the value" of a given carrier's property, so that, if a charge of confiscation through rates which are too low is made, the Government will be armed with the facts and will not be dependent on the evidence of value furnished by the carrier, and such, perhaps, is the purpose, or one of the purposes, of the Valuation Law. Manifestly, if such be the purpose, the only value that will be worth anything to the Government—State or Federal—if the charge of confiscation under the Constitution be brought against it, is "the value" of the property. Any figure admitted to be less than the true value of all the property of the carrier, will be worthless, as long as the Constitution stands, and, therefore, labor and cost now expended to obtain such a figure are time and money thrown away.

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There may be a relation between the total net earnings of a common carrier and the total value of all the property of that carrier, speaking always of value as actual value, but there is not, nor can there be, any relation between the total value of all the property of a carrier and one of the individual rates of that carrier. The carrier may be earning a total that furnishes unusually large dividends and yet the particular rate involved may be unduly or unreasonably low, or necessarily fixed by competitive or other conditions.

On the other hand, a carrier may not be earning any dividends, it may not be paying all its fixed charges, and yet the particular rate at issue may be unreasonably high. The reasonableness of the individual rate must be determined in each instance on other considerations than the value of the carrier's property. Under the law each individual rate must be "reasonable", and whether the roadbed of a carrier cost \$10 000 or \$200 000 per mile to construct, and whether the carrier claims or proves its property to be worth one sum or another, it can never exact under the law any more than a reasonable charge for each particular service that it performs. Thus is the public protected against supposed or actually inflated values, if made or claimed by the carrier. On the other hand, the carrier is entitled, under the law, to make a reasonable charge for each particular service that it renders, so that, regardless of any theoretical, imagined, or depressed value that may be put on its property, it may continue to demand a reasonable charge for each of its services, and thus it is protected. It is true that the term "reasonable" is an elastic one, and throughout all the years in which various commissions have been applying it, no uniform rule for its application has ever been established. This does not mean, however, that it has not been applied, nor that eventually more general rules for its application will not be evolved. Heretofore, each individual rate case has been determined on its own peculiar facts, and such will be the situation after the carriers have all been valued and until such time as uniform rules may be worked out. When worked out, and in the process of working them out, it will be found that, save in the case of a body of rates attacked as confiscatory, the value of the carrier's property will probably cut no figure; but whether or not this be true, it will certainly be found true, as long as the present Constitution exists, that in the making or unmaking of railroad rates—either as a whole or individually—by Government decree, the only value of the property of the carrier that can be considered is its actual value, or, in other words, "the value" authorized and directed by the law to be obtained.

Judge Prouty, in his speech before the United States Chamber of Commerce, to which reference has already been made, used this language:

"The rates of public utilities are at the present day usually fixed by Commissions, both State and Federal. It is perhaps the natural

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inference that when the value of the property has been determined and the rate or return fixed, the work of the Commission in establishing the charge of the public utility is comparatively easy. It is only necessary to multiply the value by the rate and to allow a charge which will yield that income. And this, with some important qualifications, is true as to certain kinds of public utilities. Take, for instance, a water plant or a gas plant. This serves a single community. As a rule it meets no competition in that service. The amount of service is known or can be forecast with reasonable accuracy. Even matters of depreciation and such like have come to be pretty actually understood. It is possible, therefore, to fix with some confidence the rates of such a utility, when the value of the investment is known. With the railroad, however, this is entirely different for the reason that it seldom happens that a single railroad can be considered by itself. The greater part of the business of the railways of the United States is subject to competitive conditions of one sort and another, which are largely controlling so that the rates of one are necessarily bound up with those of another".

Also:

"The railroads of this country are so bound up together that their rates are largely interdependent. It is impossible to shake a single railroad free from every other and fix its charges upon the basis of a fair return upon its fair value as you would in case of a gas or water plant."

Also:

"While, however, I wish to make it perfectly plain that the problem of establishing railroad rates will not be solved by this valuation, I desire to say with even greater emphasis that the problem will be enormously simplified. It can be known with certainty whether the general level of rates is or is not too high and in establishing the charges to be observed by a single carrier, even in fixing the rate upon a single commodity, it will be of much benefit to know the value of the property involved".

All of which means that, in the opinion of Judge Prouty, and as any serious consideration of the subject will show, there is not in law or reason any such thing as a special value different from real value on which to base railroad rate-making, and that the only value of use or service in any degree in the testing or making of rates is "the value" of the property. Judge Prouty, in making rates for a gas or water plant, would want to know "the value of the investment" and not some fanciful and untrue value based on a theory. In the case of railroads, as we have heretofore seen, he construes the law to require "the value" of the property to be ascertained, and now we see that when "the value" has been ascertained, he cannot solve the problem of rate-making on that value. The most he can say is that, knowing "the value", he can know with certainty whether the general level of rates is too high or too low, and that in other respects it will be of much benefit to know "the value" of the property. What this benefit will

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be he does not say, but, accepting his statement that there will be a benefit, it is important to note that the benefit will come from knowing "the value" of the property. The problem of railroad rate-making is an involved and complicated one. Many and conflicting elements enter into it. With such a problem to solve, it should require no argument to demonstrate that injecting into it a fanciful and untrue value of the property involved, not only adds nothing to the solution of the problem, but actually destroys any test of whether the general level of rates is too high or too low.

On page 9, the Committee states that in examining accounts to secure costs of a property that is not old, "it will be necessary to scrutinize the accounts and to examine the property to ascertain whether the construction was carried out in a judicious manner. * * * whether the cost was reasonable".

The writer submits the opinion that, without living witnesses, it is doubtful whether disapproval of properly kept accounts would or should stand when made by any outside party not personally acquainted with the property and its history.

Page 10 of the report is evidently written with the erroneous idea that maintenance, replacements, depreciation, and obsolescence of a railroad, say, 50 years old, has not been generally covered by charges to operating expenses or from income; and, further, under the assumption that on ordinary railroad properties care has not been usually taken to charge to capital account not more than the true betterment or addition. The Committee is in error in making such a sweeping assumption the basis of one of its important conclusions adverse to the records of railroads.

The law directs the Commission to report "original cost to date" of all the property of each carrier, and the duty of the Commission to ascertain and report the figures representing such cost is mandatory; notwithstanding this fact, however, the Committee, instead of offering helpful suggestions for the discharge of the duty, ignores the requirements of the law and on page 11 of its report says: "For reasons given, the Committee believes a valuation of an old property based on its actual cost to be generally impracticable, making it necessary to adopt the cost-of-reproduction method".

On page 15 of the report, the Committee says:

"In view of these and other considerations, the Committee is of the opinion that valuation should be based upon the physical conditions existing at the time the various portions of the property were built, but at the prices prevailing at or near the time of the valuation".

It is to be observed (*a*) that the Committee, ignoring the law under which railroads must be valued, gives no consideration to "original cost to date", which the Commission must ascertain and which will show what it actually cost to construct the property under the condi-

tions existing at the time of construction; and it is also to be observed (*b*) that the Committee is clearly influenced in its suggestion by the "purpose" for which it would get a certain kind of value; for certainly it would seem that if the physical value of a certain property is the cost of reproducing that property, then the cost of reproduction to be ascertained is the cost to-day, for it is the value of the property to-day —at the time it is being used—and not a value existing years ago, to which the protection of the Constitution extends. For all purposes of value it is the value "to-day" which is involved, and which must be shown. Mr. Churchill.

On page 18 the report reads:

"Although the Committee believes that in such cases as those cited the property to be valued is that which the corporation has paid for rather than that to which it holds title, there are obviously many cases in which property donated to the corporation should be included in the valuation."

On page 50, it reads:

"Your Committee is aware that the decisions of the higher Courts indicate that present prices should be used in making a valuation of land as well as of structural property, and that the present practice generally gives to the corporation as to the individual the so-called unearned increment due to appreciation in land values.

"This is incompatible with the fundamental principle that the corporation is entitled to a fair return upon the fair value of its property, because, if the land is valued at the increased price, and the appreciation in the value of the land is included in the accounting, the corporation would receive a sum in addition to a fair return, amounting to the appreciation in the value of the land."

On page 74, it reads:

"*Adaptation and Solidification* should be included in the valuation to the extent that they may have involved the expenditure of money. The increased value due to adaptation and solidification which has resulted from the action of the elements and natural causes without the expenditure of money should not be so included."

These quotations, and others which might be made, show the intention of the Committee deliberately to exclude from its valuation of a carrier's property certain distinct items and elements of value, so that, as we have heretofore seen, the result of a valuation reached under its recommendations would confessedly not be "the value" of the property. Not being and not pretending to be "the value" of the property, of what service would its valuation be? So far as a given railroad property is concerned, it has already been shown that given the true actual value of all its property, tariff rates for it could not be predicated thereon. This being true, what could any one possibly do with figures not representing "the value" of the property?

Mr. Churchill. The error of the Committee in making its report is fundamental. On page 6, it is said:

> "It has been well settled by the highest Courts that the owner of such property [public service property] is entitled to a fair return upon a fair value of the property utilized in or reasonably necessary to the service."

Without at this time entering any dissent to this statement, attention only is called to the fact that throughout the report, the Committee has made the fundamental error of misconstruing and misunderstanding the basic principle of its report as thus stated. In other words, the Committee, as clearly appears, has proceeded on the assumption that "fair value" means something less than real value, and, so proceeding, has sought to work out a rule, by which, in valuing property, a figure may be arrived at, which, although less than "the value" of the property, will be its "fair value". The Committee, departing from substance, has wasted its energies on shadows; for "value", "fair value", and "reasonable value" mean the same thing, under the Constitution. If, however, by a play on shades of meaning, it is held that "fair value" is something less than "value", then only the figures representing "the value" are of any use when the Constitution is invoked. "Fair value" means no more and no less than "the value" fairly ascertained. The use of the word, "fair", may imply a limitation to the extent that "the value" required shall not be "fictitious value" nor "excessive value"; but it is likewise a limitation to the extent that "the value" required shall not be less than "actual value". The value required is not that which is fictitiously or excessively high or low, but is the real value fairly and reasonably ascertained.

In the Minnesota Rate Cases (230 U. S.), referred to by the Committee, the Court, quoting from Smyth *vs.* Ames (169 U. S., 546), said:

> "The basis of calculation is the 'fair value of the property' used for the convenience of the public", but proceeded immediately to add: "Or, as it was put in San Diego Land & Town Co. *v.* National City [(174 U. S., 739)]:
>
> "'What the company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used for the public.'"

Thus, the Court clearly implied that "fair value" and "reasonable value" were, for its purposes, synonymous terms. A few lines farther on in the opinion of Mr. Justice Hughes, from which these quotations are taken, he proceeds to quote with approval a further statement from Smyth *vs.* Ames, to wit:—"What the company is entitled to ask is a fair return upon the value of that which it employs for the public convenience". Thus, "fair value", "reasonable value", and "value" are used by the Court as interchangeable and synonymous terms. Later, in the opinion of Mr. Justice Hughes in the Minnesota Rate Cases, he

uses the following language: "It must be remembered that we are concerned with a charge of confiscation of property by the denial of a fair return for its use; and to determine the truth of the charge there is sought to be ascertained the present value of the property." Thus emphasizing again that it is actual value that must be ascertained. Mr. Churchill.

The law requiring the ascertainment of "original cost to date", which requirement the Committee ignores, proceeds to say:

"Fourth. In ascertaining the original cost to date of the property of such common carrier, the Commission, in addition to such other elements as it may deem necessary, shall investigate and report upon the history and organization of the present and of any previous corporation operating such property; upon any increases or decreases of stocks, bonds or other securities, in any reorganization; upon moneys received by any such corporation by reason of any issues of stock, bonds, or other securities; upon the syndicating, banking and other financial arrangements under which such issues were made and the expense thereof; and upon the net and gross earnings of such corporations; and shall also ascertain and report in such detail as may be determined by the Commission upon the expenditure of all moneys and the purposes for which the same were expended."

The Committee, in its report, page 65, says:

"It is the view of the Committee that an excess of deficiency of past earnings of an unregulated property should not be considered in making a valuation of the property, except as they may be useful in determining the development expenses. The Committee has been led to reach this conclusion, not because it would necessarily be inequitable to consider such past earnings, and especially recent earnings, but there seems to be no rational basis upon which they can be included. They are important as indicating whether rates are too high or too low.

"If one attempts to consider past earnings in making a valuation, many questions arise, Shall the consideration include the earnings for 2, 5, or 10 years, and if so, why not for a much longer period?" * * *

One has but to contrast this view of the Committee with the law to which it applies, to see how hopelessly apart the Committee and the law are. The law requires all "earnings" and all "expenditures" to be ascertained. The Committee say these things should not be considered. The law would have them considered in ascertaining "original cost to date", but the Committee, as we have seen, would not obtain that cost. The "excess or deficiency of past earnings of an unregulated property" will be considered by the Commission, because the figures will be ascertained and will be of a period, in many cases, when there were no regulating commissions. Of what value, therefore, is the recommendation of the Committee that the Commission must not do that which the law says the Commission must do?

On the subject of depreciation, the report deals at length and most interestingly. There is much said in this connection which is of value and will be helpful hereafter in solving this problem. There runs

Mr. Churchill. throughout all that is said, however, the same fundamental error which pervades the entire report, and that error is the purpose to obtain a figure for "value for rate-making", in lieu of one that will be "the value" of the property. So understood, the Committee's recommendations for ascertaining depreciation cannot be considered. The law requires that, as to a given existing piece of physical property, there be ascertained the cost to reproduce it new, and that there be then ascertained the extent or amount to which that existing item has deteriorated from new, which will be the depreciation, and which amount, being deducted from that representing the cost new, will give the present value of the existing item as it stands. This is a simpler statement of what depreciation is than the process for determining its amount may be, but it is a statement wholly unsatisfactory to the Committee.

On page 33, the Committee sets up what it calls an Equal-Annual-Payment Method, which, it acknowledges, it is not feasible to comply with exactly; and, on page 34, it is shown that it is equal only because the Committee has shrewdly chosen for illustration property of 20 years' life, and an interest rate of 5 per cent. It is shown on the same page that it is not equal in any other combination. It is acknowledged, on page 36, that "it is impracticable to devise any rule which will provide for strictly equal annual payments if the cost of repairs and operation is included".

Yet this is just what railroads have successfully done for years with fairly average annual expenditures covering repairs, replacements, renewals, and including the various forms of depreciation. Hence, why should the Committee endeavor to set up a scheme primarily calculated for local water-works, pipe lines, etc., and try to make it apply to railroads which have been purposely designed so as to allow of economical replacements of their elements in such a manner that the railroad property as a whole may always be kept practically at about 100% efficiency and value? It seems that the Committee in seeking methods of bookkeeping has laid aside the practical and correct methods which have been in use for years, and which it has failed even to attempt first to disprove.

The report, page 32, says:

"A depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

On page 37:

"Assuming the Committee's views to be correct, that a depreciation allowance is a return to the corporation of a part of its investment in existing items of property, the money so returned should be treated as a part of the capital and used for any purpose for which the corpora-

tion is authorized to use capital, namely: for additions, replacements, betterments, or the extinguishment of outstanding obligations." Mr. Churchill.

On page 38:

"Depreciation allowances should be such that the corporation shall have received, when its property is worn out or become obsolete, neither 50 nor 150% of its value, but as nearly 100% as the best methods of determining the depreciation allowances will permit."

On page 49:

"It will be observed that the writers in all these cases ignore the fact that a depreciation allowance earned from the rate-payers is a return of a part of the investment, and that after that part of the investment is returned, it is neither inequitable nor confiscatory to base future returns upon the depreciated value, which is the remaining investment in the property."

It seems quite clear from these quotations, and from others that might be made, that the purpose of the Committee in dealing with depreciation is to obtain what in its judgment will be a fair "depreciation allowance", and not the actual existing depreciation as affecting present value; and that the end to be reached is not the value of the property in its depreciated condition, but the "remaining investment" in the property, after the original investment has been depleted by whatever depreciation allowance has been "earned from the rate-payers". Interesting as all this may be, can any one see any connection between it and "the value" of the property at the time it is being used for the public?

On page 6, the report reads: "The subject is clearly in the development stage, and any conclusions reached at the present time must be considered as tentative," and, on same page, it again reads:

"It is recognized, however, that the whole subject is in a development stage, and while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

The question is here presented as to the value for any purpose of views which admittedly are but tentative, and which, in addition, are concededly in violation of the law as declared and expounded by our highest Courts.

It is to be said in conclusion that though the report as a whole clearly indicates that it was written with reference primarily to such public service corporations as water and gas plants, it nevertheless contains much that is of value in the valuation of the property of common carriers. It discloses much thought on and consideration of the valuation problem, and contains many suggestions that will

Mr. Churchill. be helpful in the solution of this problem. It is so fundamentally wrong, however, in the theory on which it is prepared, that as a whole it has no value for those dealing with the property of carriers. As it is written, a careful and exact following of its recommendations would not result in any one knowing "the value" of the property appraised. The report designedly and purposely excludes "value" as an end to be obtained, and substitutes therefor a sum, arrived at by arbitrary methods, to be used as "value for rate-making".

If it was desirable for such a sum to be found, the suggestions of the Committee as to methods for arriving at it would be entitled to due consideration, though, of course, it would have to be admitted that the fixing of this sum, whatever it should be, is more of a Governmental than an engineering problem. No purpose, however, could be served by a special value for rate-making as applied to carriers, because "the value" of their property when known will not solve the problem of rate-making, either for them or by them.

The Committee's report ignores (*a*) the requirement of the law that "value", without regard to the purpose of its use, be ascertained; (*b*) that the "value" when ascertained may be used for many other purposes than the regulation of rates; (*c*) that even in rate regulation it is only "the value" of the property that can be used; (*d*) that the protection of the Constitution extends to "the value" of the property; and (*e*) that when in support of a charge of confiscatory rates, a carrier presents the figures showing "the value" of its property, the Government cannot defend by showing a "value for rate-making" which is admittedly less than "the value" of the property.

For these reasons, and because the railroads of the country must be valued under the law, and not in accordance with any man's theories of what the law and the Constitution should be, the report should be referred back to the Committee to be harmonized with the law and the facts; and as this Committee was appointed "to formulate principles and methods for the valuation of railroad property and other public utilities", many portions of its present report should be revised, especially its Summary, Paragraphs No. 1, "Report Limited", etc.; No. 2, "General Principles"; No. 14, "Basis for Rate-Making"; No. 16, "Equal-Annual-Payment Method"; No. 19, "Appreciation"; and also the details of some other paragraphs. Further, these summary paragraphs should not be written as conclusions. On the other hand, as heretofore stated, the whole report should be changed so as to make it one of progress, from an engineering standpoint, and the Committee should be continued so that it may keep in touch with the reports of valuation, and with decisions of the Courts, as they are made from time to time, bringing to the attention of the members of the Society new features as they develop on this great subject, which, as the Committee states, is so clearly in the development stage.

M. H. BRINKLEY, ASSOC. M. AM. SOC. C. E. (by letter).—The Committee's report is a very comprehensive expression of valuation principles. The Equal-Annual-Payment Method of computing depreciation is both practical and logical. However, the decision of the Committee in regard to the use of depreciated value for rate-making may be supported by other considerations in addition to those given in the report. Mr. Brinkley.

In accordance with the agency theory of corporate control, the true relation existing between the public and the utility is that of principal and agent. The utility has a franchise which, in effect, is a contract between the two parties. In the very nature of the franchise, the utility has the right to do certain things which are ordinarily only vested in the State. The regulatory power of the State over rates follows naturally, because the utility is its agent.

The Courts have uniformly held that rates shall be reasonable, taking into consideration the value of the property. By "value" is not meant capitalized income, for if value is based on rates and rates on value, rates are always reasonable. Being denied the use of capitalized income, which is the real value of the unregulated property, the use of the actual investment or investment cost is the next step. The use of investment cost follows naturally from the agency theory.

If the utility is the agent of the public and the regulatory power is exercised from the beginning, the actual investment is the value on which rates can be predicated. However, the investment cost is not often obtainable, and, in lieu thereof, physical value is taken, as the investment was the physical value when the utility was new.

The physical value will have depreciated with the age of the utility. It must be assumed, however, that repayments have been made, either by unwarranted dividends or through earnings being placed in additions and betterments.

Practically all the utilities in the past have been unregulated. The adequacy of rates for the repayment of capital on account of depreciation has been under their control. If they have not been repaid, is it the fault of the public? The last paragraph of the Committee's quotation from the Knoxville water case is very apt in this connection.

If dividends by good management could not have been sufficient for repayment of lost value through depreciation, then the stockholders have simply made an unfortunate investment, as other utilities are not condemned for paying excessive dividends. Unearned dividends cannot be capitalized, and this applies to dividends which are insufficient to repay loss due to depreciation.

It has been stated that depreciation is a liability of the stockholders. Then, if true, replacements should be paid for out of dividends. Permanent depreciation has been called deferred maintenance, but the company will never again need to go to the expense of placing the property

Mr. Brinkley. in 100% condition, because the life of all items is not the same. In past years there has been depreciation without any corresponding maintenance.

The simplest illustration of value is sales value, or value in the markets of the world. In the sale value of any object or unregulated property, the condition or depreciation is taken into account. The revenue being derived might also be considered, but the foundation value or physical value is the depreciated or present value. In the present value, the appreciation of some elements, as well as the depreciation of others, is accounted for. Justice is certainly done if they are both considered.

In basing rates on present value, the utility is not simply viewed as an aggregation of inert physical elements which have depreciated with age. On the other hand, it is seen as an operating property which is yearly taking out of earnings amounts sufficient to repay capital for depreciation. It is repaying this uniformly with the least drain on the earnings of the utility. The Straight-Line Method does not do this, as shown by the Committee, as the earnings have to be larger in the earlier than in the later years; but the Equal-Annual-Payment Method brings about the repayment in the most efficient manner, because it takes place on a compound-interest curve. The earnings on the remaining investment are the complements of the yearly repayments, and their total amounts will be the same each year if the interest rates are the same. As it happens, the actual depreciation takes place approximately in the same way. It is small in the early years of the life of the items and increases yearly to the end. It has been stated that, on account of the fact that railway companies are in 100% operating efficiency, they should be valued at cost of reproduction, which in this case means reproduction value as generally used. Does 100% operating efficiency mean that a utility structure is from 0 to 100% more valuable than one of the same age, life, and original cost which is being used for other than public service purposes? If there is any such additional value, it is a non-physical or intangible value, due to its use by a utility. In that case, however, the non-physical value would always amount to the actual depreciation. This value would then be greater in an old structure than in a new one, which is absurd.

In regard to operating efficiency, it must also be stated that we are not making a valuation of the efficient management of the owners, but are estimating the actual physical value of the property, and, in finding the physical value, the operating efficiency cannot be considered. Operating efficiency should only be considered in the rate of return allowed on the investment. It should there be given due consideration.

The advoctes of reproduction value for rate-making purposes contend that it is the nearest approach to original investment; but they fail to take into account that the property has been operating since

Mr. Brinkley.

originally constructed, and, in finding the investment, the average earnings to date should be considered as the property has depreciated while producing earnings. This is on the basis of the agency theory exclusively. On this basis, if right of way for railways is included at present values, its increase in value from original cost should be counted as earnings. If this is not done, it should be placed at original cost.

If right-of-way multiples for railways are used, as advocated by the Committee, they are being taken on values which are in great part produced by the existence of the road itself. It would be very difficult to calculate what the values would be if the railway did not exist. The only alternative seems to be to take it no higher than the present market value of adjoining land, as this is the only measure of value which we have. However, if the actual cost was greater than the present market value, it should be given due consideration.

In the foregoing discussion, the views set forth are only meant to apply to a property in which no depreciation fund exists. For a property which has a depreciation fund, the writer can only express agreement with the views so ably set forth by C. E. Grunsky, M. Am. Soc. C. E., in his paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates."* In that paper he showed that, if a depreciation fund existed of an amount equal to the actual depreciation of the property, the reproduction value should be taken as the value for rate-making purposes. This is shown to be consistent with the use of present value, where no depreciation fund exists, as was shown by the writer in his discussion† of the paper by William J. Wilgus, M. Am. Soc. C. E., on "Physical Valuation of Railroads."

For a property with no depreciation fund, reproduction value could be used with the same result on earnings and rates as the use of present value, if the cost of replacements to the extent of interest on the depreciation is paid out of dividends. The net return to the stockholders is the same in the two cases, but the rate of return is different on the different values.

As stated by the Committee, development expense starts after the utility begins to operate. It cannot be considered in reproduction value new, because it is not incurred during construction, but during the early years of the plant's life, after it begins to depreciate. To a certain extent, it offsets depreciation in calculating present value. It could be included by allowing liberal value for interest and commissions.

When one contemplates the rate regulation problems, and the adjustment of rates with physical valuation, as the main basis, anything but an approximate solution for competitive utilities, such as railways,

* *Transactions*, Am. Soc. C. E., Vol. LXXV, p. 770.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 244.

Mr. Brinkley. seems hopeless. Under the circumstances, public ownership of railways and other utilities appears to be inevitable at some time in the future. If it comes, it will not be on account of the possibility of lower rates, but the desire of the public to have better control of public agencies, whatever the cost.

Mr. Stone. H. M. STONE, M. AM. SOC. C. E. (by letter).—The title of the report as submitted is "Valuation for the Purpose of Rate-Making", while the Committee appears to have been appointed "to formulate principles and methods for the valuation of railroad property and other public utilities".

It should be distinctly borne in mind that methods of estimating depreciation on gas, water, and electric light plants, or similar public utilities, are not the same as those used for estimating on railroads. The railroad property is in the open, can be readily examined, and is not allowed to run until it is worn out, but is constantly in the process of renewal. In gas, water, and electric light plants, there may be no competition; they serve all the requirements of a community, quite in contrast to railroads which have to furnish equal service at the same price, and in nearly all cases are strongly competitive.

Railroad rates are not based on the value of the property used to render the service, and it is well understood that such rates are based, or substantially depend, on many things, among which may be mentioned: (*a*) value of service; (*b*) competition between carriers, localities, and commodities; and (*c*) cost of service. Carriers between competing points must maintain the same rates, and no shipper, locality, or class of traffic can be given any undue advantage or be subjected to any undue disadvantage. Rates, from different sources of production of the same or like commodities to the markets therefor or to the places where the same are consumed, must necessarily bear a certain relation to each other.

The report of the Special Committee ignores all the considerations on which railroad rates are, in fact, based, and erroneously assumes that they are, or can be, based on the value of railroad property.

The report is only a discussion of valuation "for the purpose of rate-making" (page 70). It asserts in substance that the depreciated value of the property should be the basis for rate-making, and proceeds on the theory that the depreciation allowance is in fact a repayment in installments by the rate-payer to the corporation of a part of the investment in particular items of property. It seems to be the opinion of the Committee that the sum said to be so repaid should not thereafter appear in the valuation of such items.

Applied to the valuation of a public service plant, like a waterworks, owned by a municipality, or enjoying an absolute monopoly, the report contains suggestions that may be useful in the work of

appraisal or in determining what rates, under the circumstances, ought to be fixed, but it is utterly inadequate as a guide for the work of ascertaining the value of railroad property, and the writer believes that certain statements therein are imperfect and misleading. Mr. Stone.

The Act of Congress directing the Commission to value railroad properties does not specify the purpose of the valuation, and any assumption that it is "valuation for the purpose of rate-making" is not justified.

It is a mistake to assume that either the cost of reproduction new or the cost of reproduction less depreciation may be used as a measure of, or sole guide to, the value of railroad property. The Valuation Act requires the Commission to investigate and report the value of all property owned or used by the carrier, and directs it to ascertain and report in detail, as to each piece of property, certain facts which are, or may be considered as, evidence bearing on value, among which are: (*a*), the original cost to date; (*b*), the cost of reproduction new; (*c*), the cost of reproduction less depreciation; (*d*), other values and elements of value; (*e*), the history and organization of the present and of any previous corporation operating the property; (*f*), any increases or decreases of stocks, bonds, or other securities; (*g*), the moneys received by reason of any issues of stocks, bonds, or other securities; (*h*), the syndicating, banking, and other financial arrangements under which issues were made, and the expense thereof; (*i*), on the net and gross earnings of such corporations; (*j*), the expenditure of all moneys and the purposes for which they were expended.

The writer is of the opinion that no one of these items of evidence is to be taken as the measure of the value of railroad property, but that it is expected that the Commission will take all into account and give to each its proper weight in the ascertainment of the value of the property.

The report (page 6) proceeds on the theory that "The valuation is for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more". Rates may be fair and reasonable and still be high enough to yield a substantial surplus after the payment of operating expenses, including maintenance and proper allowances for depreciation, and after the payment of a fair return on the full value of the property. As a general rule, railroad rates ought to be high enough to permit the accumulation of such a surplus in order to establish and maintain credit. In some instances, however, reasonable rates may not be sufficient to yield a fair, or any, return on the values of the property; but such instances are rare, and are usually due to peculiar or temporary conditions. The value of railroad property is not a means for the ascertainment of the reasonableness of rates, and, as a general rule, is not used as a guide by which rates are fixed. A railroad company is

Mr. Stone. entitled to a fair return on the value of the property used in the public service, if that return can be earned by the application of rates which are not unreasonably high, and it cannot be compelled, without a violation of constitutional rights, to put in rates so unreasonably low that confiscation will result. The only cases in which the value of railroad property or the rate of return thereon is of importance as affecting rates are those in which it is claimed that Governmental authority has prescribed rates which are confiscatory.

It is always to be borne in mind that a rate may be sufficiently high to withstand attack on the ground that it is confiscatory, and yet be so low as to be unreasonable and contrary to a wise public policy.

On page 7 of the report it is stated in substance that

"Justice and equity require that in any regulation of rates fixing the amount which may be earned by a public service corporation there shall be taken into consideration two distinct features:

"First, the annual return covering interest and profit to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment.

"Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

It is respectfully submitted that this statement is inaccurate, and that the allowances are insufficient, because these allowances do not provide for any margin of safety or the creation of any surplus to serve as a basis of credit. It is in the interest of the public that the credit of the carriers be sufficient to enable them to procure funds necessary to provide facilities to meet the increasing demands for service.

Depreciation, as set forth in the report, is impractical, and almost wholly inapplicable to railroads. There are no tables of mortality that will give any reasonable results, nor would it be possible to make such tables for the bulk of the items of railroad property, nor are they a factor in the determination of rates.

The maintenance department of a railroad, at the close of a year, knows very closely whether or not there has been enough expended on the property to keep it up to its adopted standard. Any system of accounting by the use of mortality tables is relatively worthless as compared with actual knowledge as to the main facts in connection with the maintenance of a property, which knowledge is in the possession of those only who have had the maintenance in hand. The same holds true of equipment, that is, those having charge of the maintenance of equipment of a road know very definitely whether or not the equipment is kept up from year to year.

Mr. Stone.

The so-called depreciation is no more or less than the amount necessary for maintenance, and that amount can be known only by knowing the facts in the case for each property; it can never be ascertained by the use of mortality tables, which would introduce a mass of fallacious accuracy that would require continual adjustment.

Depreciation is not an amount paid back to the investor, which should no longer appear in the valuation, but it is the amount that it is necessary to spend to keep the investment intact, and, therefore, in any valuation made to indicate on what amount a railroad should be allowed to earn a return, it has no place whatever.

There are some instances where, as a matter of policy, a company should provide for some extraordinary renewal which it is apparent is approaching, by some special arrangement that will not throw the burden of this renewal into a single year; but that is a matter of expediency on the part of the railroad, and is not a matter of valuation.

On page 10 of the report it is in substance suggested that the cost of all obsolete items be deducted.

Without here expressing an opinion as to whether or not it is generally right to take into account obsolescence in the ascertainment of cost of reproduction less depreciation, the writer suggests that it is clear that the Valuation Act does not require obsolescence to be taken into account in the ascertainment of the cost of reproduction less depreciation. The depreciation there specified is to be ascertained by a comparison of the state of repair or physical condition of the item of property at the date as of which the valuation is made, compared with its condition when new.

On page 13 of the report it is suggested that, "An engineer in ascertaining a value for rate-making must take into account whether or not the property, although owned by the corporation, is in use."

It is not necessary that all railroad property must be in actual use on the date as of which the valuation is made, but there should be included all property acquired and held by the company, which it is expected will be used in the future. For example, in the development of a terminal in a large city, several years may be required in order to secure the needed land and to perfect arrangements with public authorities. Property acquired for immediate use and actually being prepared therefor at the date the valuation is made certainly ought to be included. It may be difficult to state a general rule to cover this point, but it is certainly unfair to make "actual present use" the test.

The attempt to segregate land into different classes, according to its use or proposed use, cannot properly enter into a just valuation. The owners of railroad property have decided that the acquirement of certain lands was a proper course to take. To treat land donated by the Government as of different value than other land, because of

Mr. Stone. such donation, is unjust. As a rule, such land has been paid for several times over through the medium of special service rendered by the railroad to the Government. It cannot be said that some arbitrary strip of land has one value, and that another strip, possibly immediately adjoining, has a different value, basing such value on the apparent or present use.

On page 15 of the report it is suggested, "that valuations should be based upon the physical conditions existing at the time the various portions of the property were built, but at the prices prevailing at or near the time of the valuation."

The Special Committee is opposed to including in the estimated reproduction cost that of laying gas mains or water pipes in a street the expense of tearing up and replacing pavement which did not exist at the time the mains or pipes were laid, and seems to hold that such expense is no part of the present value of the mains or pipes, and apparently for that reason reaches the conclusion that the same should not be included in the estimated reproduction cost. It seems to the writer that this results from assuming that every figure which is used as "evidence of value" must be the same as that which "expresses the value". There is no doubt that in case of present reproduction the pavement would have to be torn up and replaced. Reproduction cost is not synonymous with value, but merely one fact or item of evidence to be used in ascertaining value. The engineering conception of reproduction should be followed throughout, and should not be modified to correspond with varying conceptions of value or to meet the supposed requirements of justice in particular instances. It is for the tribunal charged with the duty of finally ascertaining the value to give to the reproduction cost figure and to each of the other established facts such weight as they are entitled to in the light of all the evidence.

Again, on page 17 of the report, it is said:

"If after a pipe has been laid in a street new grades were adopted, the reproduction cost of the pipe should be estimated on the basis of the original grades, and if the regrading required a raising or lowering of the pipe already laid, the cost of such work, if not charged against the rate-payers as a part of the current expenses, should be included in the reproduction cost."

This statement confuses original cost with the conception of reproduction cost. If the property were presently to be reproduced, the cost thereof might not be affected by the fact that pipes had been raised or lowered since original construction, and it is not perceived how the source of funds expended for laying or raising or lowering the pipes would affect the present cost of reproduction.

On page 31 it is said:

"The Committee has already suggested that many principles and methods applicable to a normal property are not applicable to a 'losing

venture'. For instance, the 'losing venture' is not entitled to what would otherwise be a fair return on the fair value of the property. It has to be recognized in such a case that the investor must accept the resulting losses and such rates as the services rendered are worth, and to be recognized in such a case that the investor must accept the resulting earnings and not on other methods of valuation." Mr. Stone.

Undoubtedly, a railroad company cannot maintain rates that are unreasonably high merely because its enterprise is not a financial success, but it is difficult, even in the case of a "losing venture", to see how rates or revenue can be used logically to determine value, if that value is to be used as a basis for rates. The conception of the Committee seems to be that the rates are to be fixed at what the services are worth and that value may be based on earnings derived from such rates. This idea ought not be confined to "losing ventures". A railroad property so located, constructed, and operated that, at rates measuring the fair value of the service, it can earn an ample surplus, after the payment of operating expenses, together with a fair return on value, is certainly worth more than the cost of reproduction less depreciation, and such properties are usually worth more than the undiminished cost of reproduction new. It is unreasonable and unfair to limit the value of such a property to "The Depreciated Value" (page 72), and then to fix rates so low as to be sufficient only to pay return on such value.

The writer, of course, recognizes the fact that a railroad may be located and built so that it may be worth even less than cost of reproduction less depreciation, but such instances are unusual.

On page 32, it is said:

"A depreciation allowance is, in effect, a payment from the ratepayer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

It is always proper to include in operating expenses a sum sufficient to maintain each element of the property in proper repair and to replace its worn out parts. This "depreciation allowance" is not a payment to the corporation of a part of its investment in any proper sense, but is a just part of the operating expenses.

On page 41, it is said:

"There is a fundamental objection to depending upon the personal judgment of appraisers, as this method determines only the total depreciation of the property and not the amount of the depreciation allowance which the corporation is entitled to earn, unless it is to be assumed that the difference in the amount at which the property is appraised from time to time furnishes a measure of the amount of depreciation which should be allowed during the period between such appraisals. It is obvious that very erratic results would be obtained by such a method, especially if different persons acted as appraisers in different years."

Mr. Stone. This statement seems to fail to make a distinction between the ascertainment of the value of the property in its present condition and the amount of money required to keep up the property and to renew its worn out parts. The value of the property is one thing and the amount of money necessarily included in operating expenses to repair it and to renew its parts is another and entirely different thing—one is the appraisal of the value and the other is the ascertainment of an element of operating expenses.

On page 50, under the caption, "Appreciation", the report seems to indicate that it is the view of the Committee that railroad companies are not entitled to the increase in value of their right-of-way and terminal lands; it is said that such lands "sometimes increase in value so rapidly that the question arises as to the justice of basing rates upon the present value of the land rather than its original cost."

The writer understands it to be settled by the highest authority that, if it can be earned on reasonable rates, a railroad company is entitled to a full and fair return on the value of the property at the time it is used in the public service. It is known to everybody at all familiar with railroad affairs that, notwithstanding enormous increases in value of terminal lands, rates have constantly decreased. It is not necessary to confiscate the increase in value in order to have reasonable rates.

It is stated in the report:

"The Courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value. The principle involved in the two cases is the same, and there is no reason to think that its application to both would not be supported by the higher Courts."

"The sum necessary to offset depreciation" is just as plainly an operating expense as the cost of labor or fuel required to operate the locomotives. The present value of the terminal lands of a terminal company is property, and the company is entitled to pay for its use in public service and is not limited to a return on that portion of the value which existed at the time of acquisition. Railroad property is private property, subject to regulation to prevent excessive charges and discrimination. As the writer understands the suggestions of the Committee, it is that the appreciation in the value of land be considered as an offset and payments of depreciation of other parts of the property. It is certain that the company cannot use the increase of terminal land value directly to maintain its roadbed or to repair or replace worn out equipment or structures. The appraisal of land at present increased value and the allowance of a return thereon to take care of depreciation (page 50) not allowed for in operating ex-

penses—as it should be—is unfair, and amounts to confiscation of such return or increased value. Mr. Stone.

On page 67, it is stated:

"Embankments and the slopes of cuttings may become more stable on account of the growth of grass and weeds, and may therefore be considered to be more valuable, but such value as this, which is not the result of expenditure by the corporation, should have no place in the valuation of the property."

This ignores the well-settled proposition that the railroad company is entitled to full present value of its property. The principle on which this suggestion rests would deny to the company the value of lands and other property granted or donated as an inducement for the building of the road; it would deny to the company the value of all property created out of surplus earnings; it would deny to the company the increase in value of wooden structures, due to the rise in price of lumber. In short, it would leave it within Governmental power to limit the company's earnings to a sum sufficient to pay a reasonable rate only on the money actually contributed directly by owners to create the property, and this without any guaranty of any return whatsoever on such sums.

S. Whinery, M. Am. Soc. C. E.—This report is confined mainly to the question of valuations for rate-making purposes, and deals with a number of factors under the general head of depreciation. Mr. Whinery.

To the speaker's mind, it is a serious question whether the use of this word, depreciation, to cover a number of separate elements or factors, as is now the practice of engineers, and to some extent of the Courts, is necessary or wise, especially in so far as discussions or negotiations with public officials and the public at large are involved.

The careful observer, in these times of business and industrial agitation and confusion, cannot but note that no small part of the current misunderstanding, dissent, and distrust has its roots in the failure of different classes of society to understand each other. This is especially true as regards the ability of the business community and the public at large to grasp and appreciate the views and statements of technical men and the doctrinaires. This is often due not so much to the existence of really conflicting views, as to the fact that these views are not presented in language and terms that all understand alike. This applies to some extent to the technical forms of thought and expression used by the engineer.

In professional investigations and intercourse among themselves, it is entirely proper that engineers should resort to such technical terms as have come to be recognized as having an accepted meaning by them, but would it not be better and wiser if, in communicating with those outside the Profession, and in dealing with public and semi-public matters, they would confine their professional vocabulary,

Mr. Whinery. as much as possible, to such words and to such meanings of those words, as are recognized and understood by the public?

Take, for instance, this word, depreciation. Its ordinary meaning is loss or decrease of value or of utility; but, as engineers, we have apparently read into it, possibly unintentionally if not unconciously, meanings that are, stricting speaking, quite foreign to it.

Thus, an annuity, the real purpose of which is to create a fund to repay capital at a stated time, is not equivalent to, and may not be a correct measure of, depreciation in the primary meaning of that word. In the case of public service contracts the annuity or sinking fund should logically mature at the time the contract expires, without regard to the life of the plant. Again, cost of maintenance is not depreciation but its antithesis—a provision for preventing depreciation.

The fact is that engineers themselves are somewhat at sea as to the definition and use of the word, and differ quite widely in their methods of dealing with the subject and in the several elements which they think should be included under that general head.

The Committee would render a valuable service to the Society and to the Profession, if it would give this matter careful consideration, recommend an acceptable definition of depreciation, and limit the cost-factors that may properly be included under that name.

In the equal-annual-payment plan proposed by the Committee under the general head of depreciation, the elements that make up the annual payment are not very clearly stated. Apparently, they include a sinking-fund payment to repay capital, interest charge on the capital invested, some part, at least, of the cost of repairs and maintenance, and possibly some provision for betterments. The result arrived at by this scheme may be wholly fair and reasonable for rate-making purposes, but the non-professional man will hardly be able, without more careful study than he is likely to devote to it, to understand how the annual payment is made up, and will be likely to look on it, or any other rigid depreciation formula dealing with several elements, with more or less mystification and distrust. There is evidence that not a few engineers who have given the subject no little thought, seem to be in a like frame of mind.

It may be argued with much reason that the whole subject of negotiating and establishing rates may be handled satisfactorily without any reference to depreciation, in the complicated sense in which it has come to be sometimes used.

A franchise for a public service is usually for a long term of years. Its purpose is to secure for the public a stated quantity and quality of service during its life. This requires the investment of a sufficient amount of capital to supply that service in a satisfactory manner. The party supplying the service is entitled to a fair and reasonable compensation therefor, and the object of rate-making is to determine

Mr. Whinery.

what that compensation should be. Now any intelligent public official, business man, or citizen knows, or may be readily shown, that a fair and reasonable rate must be made up of certain elements, the principal ones being: Interest on capital invested, operating expenses, repair and maintenance of plant, a liberal allowance for contingencies, a fair profit above ordinary interest, and, in the end, the return of the capital invested.

There is here no mention of depreciation, nor is it necessary. The charge for maintenance takes its place. Obviously, the property must be maintained from the beginning to the end of the franchise in a condition to furnish properly the service required. If this be done satisfactorily, there can be no functional depreciation, or, at least, none from any proper point of view of the party receiving and paying for the service. The intrinsic or market value of the plant may fluctuate from time to time, as machinery is worn out and replaced, but these fluctuations do not affect the value of the plant for the purpose for which it was provided, nor call for any change in the rate of compensation, and they do not, therefore, interest the party receiving the service.

Notwithstanding, however, that the plant must be maintained in a condition to supply the service up to the very end of the contract time, its intrinsic or market value, at the end of the period, will be less than its original cost, and the difference between these two sums is depreciation in the sense in which the speaker would suggest that the word be used. Obviously, in the case of a perpetual franchise, there would never be depreciation in that sense, but the charge for maintenance would be sufficient to keep the property up to the necessary standard of adequacy.

Of course, the person, company, or engineer negotiating to supply the service may use such depreciation formula and other methods and devices as he thinks proper in computing the cost of maintenance, and must be prepared to translate his conclusions into facts and arguments comprehensible to the business man, but in all other respects the transaction will rest on plain business principles and practices which the average business man will readily understand.

The Committee seems to hold that the rate charged should include some provision for betterments and improvements. It will be difficult to convince the alert, clear-thinking, business man that such a provision is just. He will reason that if the equipment as originally designed and provided is adequate to supply the service contracted for, and is thereafter properly maintained, the cost of such maintenance being provided for in the rate, no additional capital investment will be needed to meet the full requirements of the contract; and that if additional or more expensive service becomes necessary or is wanted, supplemental contracts or franchises in which the question

Mr. Whinery. of additional capitalization and, if necessary, a proper readjustment of rates can be considered, would be the proper and business-like way to deal with the matter; and that, as to improvements, they will not usually be made unless they promise to decrease the cost of production, and consequently increase the net profits to an extent that will make the improvements, in the opinion of the contractor, advantageous to him, under which conditions the party receiving and paying for the service should not be called on to bear their cost. In any event, it would be difficult to convince him that these items have any logical connection with depreciation.

What has been said assumes that we are starting with a new project and plant, the cost of which (*i. e.*, the capitalization) may be ascertained and demonstrated with reasonable accuracy. In the majority of cases, however, where the question of rate regulation comes up, an existing plant is involved, and the question of its value, as indicating the amount of capital invested, must be determined by an appraisal, and it may be said that in all such appraisals the factor of depreciation is so important as to be controlling; but, is this true? Does not the engineer or appraiser of wide knowledge and experience actually base his judgment on concrete facts and conditions rather than on deductions from any theory of depreciation?

In judging of the value of a machine or a system of machines, the real question is not what is its age or what work has it already done, but what work is it capable of doing in the future? We want to determine its expectancy of useful life from this time on, rather than its fair average expectancy of life when new, in order that we may subtract its present age from its estimated total life and assume the remainder as its future potential useful life. Its history may be useful to us in arriving at conclusions, but such evidence is of secondary value. It is useful to know that the average life of a certain kind of machine is, say, 20 years, but we are likely to make serious errors if we use that figure as a measure of the life of all such machines. That life should be measured in units of work and endurance rather than in units of time. This is all the more true because the element of proper maintenance is very important. We all know that, of two machines, duplicates in all respects, difference in care and maintenance under the same work may make the useful life of the one 100% longer than that of the other. How then can we be justified in applying any rigid rules of depreciation based on assumed or average life to individual machines? Must not the value of each be judged on the present evidence of its probable efficiency and durability, under the conditions of service to which it is likely to be subjected? We must remember, too, that unforeseen conditions of inadequacy or obsolescence may appear at any time, in defiance of all formulas,

and the weight to be given to their probability must be purely a matter of good judgment. Mr. Whinery.

The speaker's argument is not, however, that the appraiser may not consider the bearing of any theory or formula of depreciation, or any other data or evidence available, in arriving at the value of any machine or plant, though he believes the more experienced and capable a man becomes in appraisal work the less weight will he attach to theoretical or hypothetical considerations and the more will he rely on tangible practical evidence. The speaker, however, is convinced that reports of appraisals that are to go before non-professional men, such as public officials and business men, are not strengthened by references to or discussions of complicated theories or rules of depreciation and conclusions that seem to be based thereon.

On the whole, it seems to the speaker that engineers have built up around this word, depreciation, in so far at least as it is concerned in rate-making, a complicated and unnecessary body of theory and speculation, not very useful to themselves, but well calculated to be a stumbling block in their dealings with the public.

Mr. Henry.

PHILIP W. HENRY, M. AM. SOC. C. E.—This report is an interesting and valuable document, and the Committee is to be congratulated on the thorough manner in which it has dealt with this important subject, covering as it does so many of the elements which enter into this problem. The discussion on depreciation, however, is not clear, and the speaker is at a loss to know whether the Committee intends to express the opinion that, after a depreciation reserve is set aside, the investor is entitled to a full return only on the original investment less depreciation reserve, or whether he is entitled to a full return on the original investment. As various statements on this point are apparently not in accord with one another, it would seem that the Committee should express itself with greater exactness, for it must be remembered that any report adopted by this Committee will carry great weight, not only among engineers who are more or less familiar with the principles of depreciation, but also among public service commissioners and others who may not have made a study of the subject and who, therefore, may be misled by the apparent discrepancies in the report. That the members of the Committee had clearly in their own minds conditions under which the investor is entitled to a return on his original investment, and other conditions under which he is entitled to a return only on the "remaining value of property", is not doubted by the speaker, but these different conditions should be presented with such simplicity, conciseness, and clearness as to leave no doubt in the mind of the reader, however unfamiliar he may have been heretofore with the elementary principles which govern depreciation.

Mr. Henry. As stated by the Committee: "There is no subject connected with valuation about which there are more diverse views than those relating to depreciation." In addition to this difference of opinion which exists in regard to depreciation itself, there is also a difference of opinion as to whether the same method of depreciation should be used for private corporations, where investors alone are interested; for public service corporations, where rates are regulated by public service commissions; and for railroads, where an endeavor is being made to fix rates in accordance with physical valuation. In the speaker's opinion, the same method should be used in all cases, for depreciation is one of the elements necessary to present a full and clear statement of the condition of the property; and whether this full and clear statement is for the use of the investor or of the public service commission is immaterial. In order that the investor may know the true value of his shares of stock, he must know exactly the condition that his company is in, and unless depreciation is properly taken into account, he cannot have this full knowledge. The same is true of a public service corporation, for, in order to fix rates, it is essential to know the true earnings of the corporation, and such earnings cannot be known unless depreciation is properly cared for. Therefore, whether depreciation is set aside by a mining company, the operations of which will inevitably come to an end (say, in 20 years); or whether it is set aside by a public service corporation or a railroad (the operations of which will continue indefinitely), the principle is the same in each case, and the same method of depreciation should be used.

It will be admitted that an investor is entitled, not only to a reasonable annual return on his investment (when properly and intelligently made), but also to receive back his principal at any time, whether he receives it through the sale of his property to other parties, or whether, as in the case of an exhausted mine he receives it from a depreciation reserve which has been accumulated out of earnings, and which when the mine is exhausted (assuming no scrap value), just equals the amount of his original investment. Such depreciation reserve, when properly set aside, guarantees the integrity of the original investment, whether the money is invested in a mine or in a public service corporation. If in a mine, the investor alone is interested in knowing that his original investment is maintained at a constant value, and that the return he receives in dividends, whether 5, 10, or 50% annually, is really earned. If the money is invested in a public service corporation, not only is the investor interested, but also the general public, which, by the rates it pays for the services rendered by the corporation, is interested in knowing that the property is being maintained out of earnings at a constant value (thus assuring equally efficient service as the plant grows older) and that, in addition, there are earnings sufficient to provide a reasonable annual return on the

money originally invested. If the earnings are greater than are required for these two purposes, then the public may be justified in demanding a reduction of rates. If they are less than sufficient to maintain the property at a constant value and to pay a reasonable return on the money invested, it will be impossible for the corporation to obtain the new capital necessary to maintain the efficiency of the plant, and to make such additions as are necessary to take care of the increasing business. If, therefore, a public service commission or other authorized public body establishes a maximum annual return for a public service corporation, it is important that both the commission and the corporation should know that, after paying operating expenses, including the cost of ordinary repairs and small tools, there remain earnings sufficient to pay the authorized annual return on the original investment and set aside in the depreciation reserve each year a sum adequate to maintain the property at its original value. In other words, the amount standing at any time in the depreciation reserve, plus the value of the property at that time, should equal the original value of the property (original investment), neglecting the influence which appreciation and other elements may have on the problem so fully presented by the Committee. It must be borne in mind that in this discussion the speaker confines himself solely to depreciation, without reference to other features of the Committee's report. If it is possible to arrive at general principles governing depreciation, applicable to all classes of corporations, the application of such principles to a given corporation will be comparatively simple, due allowance then being made for the effect which other elements may have on the problem. Mr. Henry.

Strictly speaking, the only true way to set aside a sum each year for a depreciation reserve is to make each year a careful valuation of that portion of the property subject to depreciation, and the difference between the value thus established and that similarly established the previous year, is the amount which should be placed that year in the depreciation reserve. Manifestly, such yearly valuation is impracticable, and it has been generally agreed that the simplest method for determining the depreciation is to assume a life-period for each part of the plant subject to wear or obsolescence, and to set aside each year a certain sum based on such life-period. Whether the investor in the assumed public service corporation is entitled to the authorized annual return on the original value or on the remaining value depends entirely on the method used in setting aside the depreciation reserve. For purposes of simplifying the discussion, it will be assumed that the depreciation reserve is invested in outside securities, although the argument holds good in case it is invested again in the property, in betterments and additions or otherwise, earning the same rate of interest as if invested outside.

Mr. Henry.

If the depreciation reserve is maintained by the Sinking-Fund Method, by which the sum set aside each year, plus interest compounded annually, amounts at the end of the life-period to the original value, the stockholder is clearly entitled each year to his full return on the original investment. On the other hand, if the depreciation reserve is set aside by the Equal-Annual-Payment Method, or by the Straight-Line Method, by which certain sums are set aside each year, totaling (without interest) the original value at the end of the life-period, the stockholder is clearly not entitled to a return on the original value, as he either receives directly the interest on securities in which the depreciation reserve is invested, or else the property receives it as an amount available for dividends.

To make this clear Table 18 has been prepared, assuming that the property has a 20-year life-period, valued when new at $100; authorized annual return, for example, of 7%; annual interest of 7%, for example, on funds standing in the depreciation reserve, such percentage being compounded annually to the depreciation reserve in the Sinking-Fund Method; but, in the other two methods, the interest being paid out each year either directly to the investor or to the property available for dividends.

TABLE 18.

End of year.	Sinking-Fund Method.			Equal-Annual-Payment Method.		Straight-Line Method.	
	Amount set aside each year for depreciation reserve.	Interest at 7% compounded annually.	Total yearly amount to depreciation reserve.	Total yearly amount to depreciation reserve.	Interest at 7% paid out each year to investor.	Total yearly amount to depreciation reserve.	Interest at 7% paid out each year to investor.
1	$2.44	$0.00	$2.44	$2.44	$0.00	$5.00	$0.00
2	2.44	0.17	2.61	2.61	0.17	5.00	0.35
3	2.44	0.35	2.79	2.79	0.35	5.00	0.70
4	2.44	0.55	2.99	2.99	0.55	5.00	1.05
5	2.44	0.76	3.20	3.20	0.76	5.00	1.40
6	2.44	0.98	3.42	3.42	0.98	5.00	1.75
7	2.44	1.22	3.66	3.66	1.22	5.00	2.10
8	2.44	1.48	3.92	3.92	1.48	5.00	2.45
9	2.44	1.75	4.19	4.19	1.75	5.00	2.80
10	2.44	2.04	4.48	4.48	2.04	5.00	3.15
11	2.44	2.36	4.80	4.80	2.36	5.00	3.50
12	2.44	2.69	5.13	5.13	2.69	5.00	3.85
13	2.44	3.05	5.49	5.49	3.05	5.00	4.20
14	2.44	3.44	5.88	5.88	3.44	5.00	4.55
15	2.44	3.85	6.29	6.29	3.85	5.00	4.90
16	3.44	4.29	6.73	6.73	4.29	5.00	5.25
17	2.44	4.76	7.20	7.20	4.76	5.00	5.60
18	2.44	5.27	7.71	7.71	5.27	5.00	5.95
19	2.44	5.81	8.25	8.25	5.81	5.00	6.30
20	2.44	6.38	8.82	8.82	6.38	6.00	6.65
Total.	$48.80	$51.20	$100.00	$100.00	$51.20	$100.00	$66.50

From Table 18, it is evident that, under the Sinking-Fund Method, the investor is entitled each year to a return of 7% on the original value of $100. Under the other two methods, he is entitled to less than 7% on the original value, which apparently is what the Committee means by "the corporation must accept a return upon the depreciated value of such property". Referring again to Table 18, in the twelfth year, for example, under the Equal-Annual-Payment Method, the investor received $2.69 from the securities in which the depreciation reserve is invested (whether inside or outside of the property), and is, therefore, entitled to receive only $4.31 ($7.00 less $2.69) from the property itself, which is 4.31% on the original value of $100. In the twelfth year, under the Straight-Line Method, the investor receives $3.85 from the securities in which the depreciation reserve is invested, and is, therefore, entitled to receive only $3.15 ($7.00 less $3.85) from the property itself, which is 3.85% on the original value of $100. Mr. Henry.

From this comparison of methods it seems to the speaker that the Sinking-Fund Method is the simplest and best, but as (to use the words of the Committee) "the use of the words Sinking Fund appears to have misled some Courts and commissions", it may be better perhaps to call this method, as above exemplified, the Straight-Line-Interest Method, which describes the method of putting aside in the depreciation reserve an equal amount each year (2.44% as in Table 18) out of earnings, and adding to it the interest on the total amount in the depreciation reserve, whether it is invested in outside securities or in the property as betterments and additions, or otherwise. As it is difficult to find safe outside investments returning more than 4%, it is preferable, generally speaking, to invest the depreciation reserve in the property itself, where it will earn the same annual return as the original investment.

From the foregoing, as well as from the report of the Committee, it is evident that depreciation is not susceptible of exact mathematical treatment, inasmuch as it is based on the life-period of the various parts making up the property, or on the condition of each part in any one year. As both the life-period and the condition of any property or of its component parts are purely estimates, on which no two engineers will agree in detail, it is evident that in setting aside a depreciation reserve, refinement is unnecessary, and the method which will best accord with present methods of accounting and which can be most easily applied is the one which should be adopted. No matter what method is adopted, and no matter how conscientiously and intelligently it is applied, it is quite likely that an experience of 5 or 10 years will show that the depreciation reserve is too great or too small, and, therefore, that a change should be made in the percentage set aside each year. For this reason it may be advisable at convenient periods to make a valuation of those parts of the property susceptible to depreciation, in

Mr. Henry. order to know whether or not the amount in the depreciation reserve equals the original value less the present value, as it should do—leaving out of consideration appreciation and all other elements which may affect the problem.

Depreciation reserve, as an account on the balance sheet, may stand alone as a liability on the credit side, or it may be shown on the debit side as a deduction from the asset, plant account. The speaker prefers the former method. No matter how it is shown on the books, or whether or not it is invested in outside securities, it should be reserved strictly for the purpose for which it has been established, and should never be considered as applicable to dividends. If it is found to be larger than conditions warrant, the amount set aside each year may be diminished or omitted altogether for a few years; if too small, the amount set aside each year may be increased.

As a matter of bookkeeping, at the end of each year profit and loss will be charged, and depreciation reserve credited, by the amount to be set aside, on each item of plant account, figuring the same percentage each year on each item, depending on the life-period and the rate of interest borne by the funds in the depreciation reserve. The depreciation reserve will also be credited at the end of each year with the interest earned during the year by the securities in which it has been invested. If it is merely a book account, and the funds have been used in the business for betterments and additions or otherwise, the depreciation reserve will be credited by interest on its average balance for the year, figured at the same rate which is being paid on the original investment, or 7% in the case which has been assumed.

On the debit side of the balance sheet, as an asset, will be found an amount exactly balancing, as an aggregate, the depreciation reserve as a liability on the credit side. If the depreciation reserve is invested in outside securities, the account on the debit side may be called "securities, reserve fund for depreciation". If the depreciation reserve is invested again in the property, it will be found, at the end of the year, that on the debit side certain assets, such as betterments and additions, accounts receivable, cash, etc., will be greater, as a total, by the exact total amount added that year to the depreciation reserve. During the year, from time to time, the depreciation reserve may be charged and cash credited (if a cash transaction) for any replacements, such as the purchase of a new engine to take the place of one which has been worn out or become obsolete; or these replacements during the year may be carried in a replacement account, and at the end of each year, the total of such account will be charged off to the depreciation reserve.

Under the Straight-Line-Interest Method (practically the Sinking-Fund Method), applied as already outlined, no question arises as to whether or not the investor is entitled to a full return on the original value of the property or only on the remaining value, as it does under

the Equal-Annual-Payment and Straight-Line Methods—thus making the latter two methods more difficult to apply. Another advantage of the Straight-Line-Interest Method is that the accounting is much simplified, and the only problem—not an easy one, however—is to establish the life-period of each item entering into plant account. Mr. Henry.

Mr. Hansel.

CHARLES HANSEL, M. AM. SOC. C. E.*—The Special Committee of the Society appointed under the resolution set forth on page 102 of the February, 1911, *Proceedings,* was instructed as follows:

"*Resolved*, That a Special Committee of seven be appointed by the Board of Direction to formulate principles and methods for the valuation of railroad property and other public utilities, and to report to the Society at the next Annual Convention."

This resolution was adopted unanimously at the Annual Meeting of the Society on January 18th, 1911. It was offered by the speaker after he had stated that, in all the papers submitted on the subject of valuation of public utilities, there had been no definite outline of the proper principles and methods to be pursued in the doing of the engineering work necessary to the enumeration and valuation of the various physical elements, and that the papers and discussions were confined principally to questions of law and economics.

In offering the resolution, the speaker felt that it was clearly understood by the members present that, as an engineering body, the Society should take up the question of valuation as it relates to the gathering and compilation of engineering data. The title of the Committee's report, "Valuation for the Purposes of Rate-Making", does not appear in the resolution, either in precise language or in terms. The report deals mainly with legal and economic questions, devoting some 47 pages to the subject of depreciation; 13½ pages to land; 2½ pages to appreciation; and about ½ page to adaptation. There is no part of the report which deals with the problems attendant upon the gathering of field data. It touches upon preliminary expenses, engineering, general contingencies, and the like; all of which are provided for in the "Classification of Road and Equipment Account" of the Interstate Commerce Commission. It seems, therefore, unnecessary to discuss the propriety of including these items, as they have already been recognized and admitted by the highest accounting tribunal in the land.

It is always desirable that the engineer broaden his knowledge through the study of legal and economic questions. However, it appears to the speaker that the first chapter of the Committee's report should be devoted to the engineering questions involved rather than to a theoretical discussion of depreciation, amortization, and other accounting theories, which are important considerations of future accounting, and not aids to the determination of the present value of the physical elements or the whole property value of a corporation which

* This discussion was presented for Mr. Hansel by Carl Tombo, Assoc. M. Am. Soc. C. E.

Mr. Hansel. has been in operation for some years. The speaker holds that there is no mathematical rule for determining depreciation, although it is quite proper that some empirical rule be adopted in accounting methods, so that the accrued replacement needs of the future may be anticipated.

In the determination of the present reasonable value of the segregated parts of a railroad or other public utility, we are not assisted by any formula of depreciation to be applied as an accounting method; and it is not likely that, in the valuation of any important railroad or other public utility, the depreciation for any purpose will be calculated by rule, but rather by investigation and observation.

The Chairman of the Committee has advised the speaker that, owing to the intricacies of the whole subject, "it was decided to take up first only one branch of the subject of valuation". It would be inferred from this statement that this report of the Committee is but one of several which it expects to make.

The Committee, no doubt, is fully advised as to the requirements of the "Federal Valuation Act", which became a law after the Committee was appointed. This Act specifically sets forth the principles and methods which shall govern the valuation of the railroads of the United States. Therefore, there is no longer need of speculation as to such principles and methods, so far as railroads are concerned. This "Federal Valuation Act" does not even suggest that the results to be obtained under its specifications are to be used for rate-making. The "Federal Valuation Act" calls for (1) "The Original Cost to Date"; (2) "The Cost of Reproduction New"; (3) "The Cost of Reproduction Less Depreciation"; and (4) "Other Values and Elements of Value". Consequently, any theory of valuation to be applied to railroad property must consider, not only the physical elements, but also such "other values and elements of value" as affect either the physical elements or the total value. Neither the Federal Government nor any of the several States has attempted to make valuations of railroad property for the purposes of rate-making. The carriers, in some instances, have found it necessary to produce evidence of the investment in the physical elements of their respective properties, in order to defend them from the enforcement of rates which they deemed would be confiscatory.

Public utilities, other than railroads, generally sell only one commodity or service, the price of which—especially in the case of gas, electric, and water utilities—is controlled by ordinance or the sovereign power of the State. Such public utilities are local, exist by the grace of special franchise, and are generally monopolistic. As contrasted with these public utilities, it is well known that railroads sell a great variety of service in competition, and are under regulation by the Interstate Commerce Commission. In the case of a railroad, there is a very great variety of rates, involving, in whole or in part, identical

stretches of the property. In the speaker's opinion, therefore, it is unreasonable to suggest that there can be a determination of each of the thousands of different rates which obtain on a carrier's property by attempting to fix the value of the physical elements of the whole property. Mr. Hansel.

It is submitted that the report is based on a fundamental error, *viz.*, that railroad rates are based on the valuation of the property of the railroad company. It will be seen, from an examination of decisions of the Interstate Commerce Commission, of the Courts, and from leading textbooks on the subject, that the rates of a railroad company are based on:

(*a*) Value of service;
(*b*) Commercial and competitive conditions;
(*c*) And in some slight degree, though very remotely, on the cost of service.

The first railroads in the United States were merely temporary paths to greatness. Modern methods of construction were not applied in the original building of any of these earlier railroads. They were built along the line of least resistance, without regard to grade or curvature. The policy of the original constructors was to build as cheaply as possible. To cheapness of construction was sacrificed shortness of line and flatness of grade and curvature. This policy, perhaps, was the better one at the time it was adopted, because the country was sparsely settled, and the most sanguine failed to grasp the growth that was to follow.

The problem with the railroads to-day is not so much how to get more business as how to carry the existing traffic at low cost. The economic demands of modern traffic require, on the part of such railroads as the Pennsylvania, the Baltimore and Ohio, the Vanderbilt Lines, the Harriman Lines, and others, that they expend enormous sums of money on improvements which are, in a large measure, merely corrective of original defects in location and construction resulting from former methods and from enforced economy due to the high price of money and the uncertainty of earnings.

The cost of operation per train-mile is approximately the same whether the train has a capacity of 200 or of 400 tons. It is evident, therefore, that—given the same rate and tonnage—the earning capacity of a railroad which, by reason of its flat grade and easy curvature, can haul 400 tons per train on a given tonnage would be much greater than the earnings of another railroad which, with the same equipment, could haul only 200 tons; and it is usually found that the railroad capable of carrying the greater train load, with a given power, costs much less to build than one of smaller train capacity. As the contractional relations between railroads influence the tonnage very mate-

Mr. Hansel. rially, it is quite possible that the cheaper railroad, having the greater train-load capacity, would be short of business on account of not being as favorably situated to receive tonnage from other roads.

There are cases where it would be economical, from a broad public policy, for a railroad to abandon its line entirely and build in different territory between two given cities, thereby saving in operating cost for carrying the same tonnage between the same termini. In many cases this saving of cost of operation capitalized would more than equal the cost of building a new line. This change cannot always be accomplished, for the reason that towns and industries must be served. Consequently, the high cost of carriage between the termini must continue; and, if a new organization constructed a line in the easier territory suggested, and secured satisfactory terminals and traffic arrangements, it is apparent that it could conduct its business at a smaller cost and with greater expedition. The securing of business, however, depends not only on convenient terminals and efficient organizations, but also on the contractional relations with other railroads which are in a position to give or withhold business; and, though the value of the physical elements of the least costly railroad, as represented by the real estate and the tangible personal property, may be much less than that of its competitors, the commercial value may be much greater by reason of its maximum train-load capacity and its minimum operating expense.

The value of low gradients and the following conditions actually exist in one of the Southern States:

"A" represents the train load of a Cooper E-50 on a grade of 15.84 ft. to the mile, or 0.3 of 1%; load, 3 022 tons.

"B" represents the train load on a grade of 53 ft. to the mile, or 1%; load, 1 322 tons.

"C" represents the train load on a grade of 60 ft. to the mile; 1.136%; load, 1 191 tons.

It is understood that the speed of the train is 18 miles per hour, the tractive force is 0.2 of the weight on the drivers, and the train resistance on the level is 6.5 lb. per ton.

It is evident that, if these three railroads have an equal division of business, and like rates, between two important cities which they connect, the net earnings of "A" will greatly exceed those of either "B" or "C". However, it is not likely that the division of business is equal, and before a final determination of value is made, the actual tonnage hauled should be ascertained; and this current tonnage is frequently controlled through contractional relations with other roads, which may vary from time to time.

If the property investment or physical value of the parts constituting the three railroad properties cited were used to determine the

justice of rates, line "A" would be given a rate about one-half of that of line "C". Mr. Hansel.

Referring to page 6 of the report: The statement that "valuation is for the purpose of determining rates which shall be a limitation of charges to those which shall give a fair return" is entirely erroneous. It has been held by the Interstate Commerce Commission and by the Courts that rates which gave more than a fair return, might be entirely reasonable; and so, also, it has been held that a given rate which affords less than a reasonable return might still be a reasonably maximum rate. The only case in which the question of return as affecting rates is of the slightest importance is where a Governmental body proceeds to establish an entire schedule of rates, and where the carrier contends that the schedule as established affords less than a reasonable return on the value of its property devoted to the public use; in other words, asserts confiscation, and accordingly a violation of its constitutional rights. If the principle be asserted that is apparently contended for in this portion of the report, that rates must be limited to such as will secure a bare return on the value of the property, it is obvious that no private capital will go into a business where the profits are limited, and where there is no guaranty against loss.

In further argument that the title is a misnomer, and that there can be no valuation for the purpose of rate-making, attention is directed to the address* of Mr. Charles A. Prouty, Interstate Commerce Commissioner, delivered at the Annual Meeting in the Chamber of Commerce of the United States, at Washington, D. C., on February 11th, 1914. Mr. Prouty is now the Director of the work involved in the valuation of railroads under the "Federal Valuation Act", consequently, his words on this subject should be carefully considered.

The principles which govern the determination of the public-service value of railroads are, in many respects, quite different from those which apply to the public-service value of other public utilities.

It will probably not be denied that railroads are the most important factor of our internal material progress. They have been constructed in advance of all other commercial developments; and, indeed, in many cases, the constructors, the bankers, and the engineers were the pioneers —the explorers of the land, at present busy with industry which now requires for its refined conveniences those other public utilities such as water, gas, and electric plants, urban and interurban transportation, and the like. Thus, the railroads stand as a necessity to the material progress of the whole country, whereas the other public utilities are local, and are created out of the demands of a refined community dependent for its being on the facilities furnished by the railroads.

The railroads differ from other public utilities in construction and maintenance. The track of a railroad is made up of small units, which

*_Railway Age Gazette_, February 13th, 1914.

Mr. Hansel. can be currently renewed; and thus the track may be, and is, generally, kept at 100% efficiency, sufficiency, and adequacy. There is, in track and roadbed, no decrepitude or obsolescence beyond the reach of maintenance.

Stated in another way, the roadbed of the carrier, which is generally a large item of cost, is constantly appreciating through consolidation, enrichment, and growth of vegetation on slopes, thus protecting it from erosion and thereby reducing the maintenance. The track itself is constantly being renewed.

A pipe line is constantly depreciating; it is beyond the reach of maintenance; when it fails, through decrepitude or insufficiency, the investment in constructing the trench, laying the pipe line, etc., is lost; and there is practically little scrap value.

Each item of track has a different life, and the history of each is represented by a series of independent, recurring cycles, which together make up the whole and produce an approximately uniform average, which, measured by efficiency, is 100 per cent.

The "Valuation Act" requires us to apply depreciation to cost to reproduce. Therefore, we must determine a method of fixing depreciation as contemplated by the Act, though we are not apprised of the purpose to which such depreciated condition is to be applied. In making this deduction for depreciation of individual items, we do not acknowledge that by so doing we depreciate the property as an instrument of commerce, which, when maintained as in the case of the majority of the carriers, furnishes a constantly adequate and sufficient system for public service.

The Special Committee was appointed under a resolution offered in January, 1911. Since then the valuation of railroads and other public utilities has been discussed by engineers, economists, and various technical societies, also before the House and Senate Committees of Congress; and, on March 1st, 1913, there was enacted a law: that an act entitled "An Act to Regulate Commerce", approved February 4th, 1887, as amended, be further amended by adding a new section, to be known as Section 19a.

Notwithstanding the enactment of the Federal Valuation Law, no mention has been made of it in the report, although it says, on page 6:

"It is obvious that a valuation before a Court or public service commission must, in order to be sustained, be made in conformity with existing laws and the decisions of the higher Courts".

The speaker desires to direct attention to other criticisms of the report, which may be stated briefly as follows:

(1) Page 7: The third paragraph, referring to the allowance sufficient to provide for depreciation, is insufficient, in that it does not provide for an adequate surplus to serve as a basis of credit.

Mr. Hansel.

(2) Page 11: The second paragraph on this page, which apparently eliminates the amount of investment as a factor in valuation, is most unjust, as it excludes entirely from consideration cost to date, and seems to assume that all investment which has become obsolete should be written off and no return allowed thereon.

(3) Page 11: The third paragraph, which recites that rates should be limited by a valuation, is absolutely impracticable in the case of a railroad company, for the reasons heretofore set forth.

(4) Page 32: There does not seem to be any justification in equity or in law for the theory apparently set out here, that the present value must exclude that part of the investment which has been replaced by amounts charged to operating expenses.

(5) Page 50: Criticism 4 applies also to the statement in the sixth paragraph of this page, with respect to appreciation and depreciation balancing each other. The statement also is apparently in conflict with the decision of the Court in the Minnesota Rate Cost, and the Consolidated Gas Case (212 U. S.); and apparently in conflict with the last paragraph on page 60 of the report.

(6) Page 67: The statement in the second paragraph, that value resulting from increased stability of embankments and slopes cannot be taken into consideration, would seem to be legally and equitably unjustifiable.

(7) Page 70: The statement in the fourth paragraph of this page is defective, in that it does not include any provision for a surplus as a basis of credit or for unforeseen expenses due to unusual destruction by the elements.

(8) Page 73: The recommendation on this page, as to treating appreciation, would seem to be neither good law nor equity. The report apparently concedes that it is not good law.

The Interstate Commerce Commission, together with its Engineering Board, Legal Advisers, and Accountants, has been engaged since June, 1913, in formulating specifications, rules, and methods of procedure for the guidance of all concerned in the valuation of the railroad properties of the United States. Up to the present time, its efforts have been directed toward the solution of engineering problems, thus illustrating the importance and difficulty of gathering engineering data. Although Commissioner Prouty, in his address before mentioned, sets forth clearly the various problems associated with the application of depreciation and "other values and elements of value", the Commission has refrained from dealing with the complex economic questions to which the Special Committee has devoted its report.

Mr. Hansel.

Any attitude, of the Federal or State Governments, toward railroads or other public utilities, which tends to throttle progress is admittedly against sound public policy. As has been stated, it is necessary for the carriers of to-day to expend large sums of money in betterments, in order to reduce the cost of transportation in the face of advancing costs for labor, materials, and supplies.

Large expenditures have been made in the reduction of gradients for the purpose of increasing the trainload and reducing the operating cost. If the beneficial results of such expenditures are to be denied the carriers, does not such action throttle progress?

The reduction of gradients frequently involves a change in gradients on the same alignment and the moving of material at maximum cost. After the accomplishment of such work, the reproduction cost of that portion of the property affected by the betterment does not represent the cost under the actual conditions of construction; and, even though it did accurately represent the cost, it seems unwise to take from the carrier the results of his efforts to increase the safe and economical operation of the property.

It must be apparent, therefore, that any attempt to fix rates of carriage by measuring the investment in the physical property is against sound public policy, even if it were possible to carry out any such plan.

Is it then good public policy to say to the carriers that it is useless for them to attempt improvements which will provide safer and more efficient transportation? Will the carriers be able to finance additions and betterments, which frequently require the abandonment of property, if they are to be denied a reasonably attractive return on the new capital as well as on the property sacrificed to the cost of progress?

It is a matter of common knowledge, that recently a carrier, which for years has enjoyed the most efficient management, has been required, by the bankers, to agree to put aside from its net surplus such sums of money each year as will produce one-half of the amount necessary to provide additions and betterments by the time the total loan is due. This plan is wise financing from the bankers' viewpoint, but it demonstrates the needs of a carrier which arise in spite of the fact that it has so remodeled its property as to put it into the first rank as to average train loading. The money which produced the present high operating efficiency must now temporarily be denied a return, because sufficient rates of carriage have not been allowed the carrier.

Shall the Society, as an engineering body, attempt to formulate principles and methods of accounting which can apply only to the future? Is it to be expected that the problems involved in the valuation of public utilities will be solved by adopting any of the several theories advanced in the Committee's report?

As understood by the speaker, none of these theories deals with facts, but rather with assumptions of conditions which do not now exist, and

can never be made to exist. All the value of all property cannot be completely measured with but one template in order to determine a proper return on capital. The term "value" is the *ignis fatuus* of the law; and, though it is defined more or less clearly in some few particular statutes of some States, it is generally left untrammeled, and free to wed with "reasonable", and, thus joined, comfort the Courts and economists, who, armed with these two indefinitive terms, are no longer in need of precise language. Mr. Hansel.

The conclusions of this Society on subjects relating to engineering are generally held in high regard, and though the opinions of many of its members, on subjects relating to law, ecomomics, accounting, and the like, are respected by many, it seems unwise, at this time, in view of the Federal legislation and Court decisions which have been uttered since the appointment of this Special Committee, for the Society to go on record as acknowledging and sustaining any formulation of principles and methods for the valuation of railroads or other public utilities for the purpose of rate-making.

Mr. Lum. D. W. Lum, M. Am. Soc. C. E. (by letter).—The Special Committee's report indicates a vast amount of labor and thought, and is very interesting as representing the personal views of the members of the Committee. The various features of the subject as treated, however, do not appear to harmonize with the practical requirements, making it necessary for the reader to choose between theory and fact in reaching a conclusion as to how to proceed in the great work that is before the country at this time.

The instructions of the Society to the Committee indicated that there was to be prepared a report to formulate principles and methods for the valuation of railroad property and other public utilities; the requirement being practically in line with that which later appeared in the provisions of the Federal Act.

As the Act was passed some months before the submission of the Committee's report, it would seem that the recommendations should conform to the requirements of the Law in order to be of practical service; and as great differences appear which cannot be reconciled, one must assume that there is error.

The writer notes with regret that the report, in its first lines, makes a most important deviation from the Act, as well as departure from the order assigning the duties of the Committee, in that it has assumed a physical inventory as the important measure of a rate-making value.

It appears proper to call attention to the fact that no reference to rate-making or earning allowance was made in the Act, nor in the order creating the Committee, and to limit the consideration in this way would defeat the object of the investigation. This view is practically expressed in the following statement by the Committee (page 67):

Mr. Lum. "There are several reasons why a physical valuation, taken by itself, furnishes an unsatisfactory basis for determining rates, and other reasons why it does not seem equitable to omit other considerations".

If, therefore, this paragraph represents a concrete expression of the Committee's views, in order that the discussion may adhere to the practical duty assigned, it would appear necessary for the report to be revised by eliminating all reference to rates and substituting principles and methods for the valuation of railroad properties and other public utilities, as described in the Act, and as called for by the Society.

The proposed untried theory of allowed sinking funds and prescribed earnings assumes so many conditions which are in direct conflict with the facts developed in the operation of a property, that one is constrained to call attention to the very familiar example as a practical demonstration of the fundamental error in the report.

Two railroads, approximately parallel, and of substantially equal length, are competing: One, reporting in accord with the suggestions of the report, would inventory physical property at $20 000 000, the other, by the same rules, would inventory $10 000 000—the more expensive road, having steep grades and adverse conditions, may cost twice as much to operate. The Committee's theory assumes that the expensive road would and must charge double the freight and passenger rate for the service, which, of course, would direct all the business to the cheaper road. The result would be that the expensive road would get no business, and would "go to the wall".

Question: If the shipper insists on using the road offering the lesser rate, even though the Committee's theory requires that the more expensive road must earn just so much each year, how does the Straight-Line Method, or the Equal-Annual-Payment Method, or any method contribute any amount in dollars to the value of the road that is not patronized. Is it not true that such a road must conform to the requirements of competition, and, on the other hand, may not the road costing the lesser amount have a value far beyond the cost of its physical construction?

If, therefore, it is not practically possible to make a separate physical "valuation for the purpose of rate-making", it would seem that the Committee should adhere to the requirements of the Law, and develop the methods and principles that are necessary at this time.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINEERS.*

By A. I. Stiles, Assoc. M. Am. Soc. C. E.

A. I. Stiles, Assoc. M. Am. Soc. C. E. (by letter).—The discussions of the report of this Committee, both in the *Proceedings* of the Society and in the technical press, seem to indicate a practically unanimous opinion that its figures and conclusions are far too optimistic. This view is supported by the writer's own experience. At the time that the information for this report was being collected, he was a member of an engineering organization employing about 20 men in the grade of transitman or higher, of whom two were members of the Society and reported their salaries to the Committee. These two men constituted the upper 10% of the entire force and their average salary was 204% of that of the remaining 90% of non-members. While somewhat more experienced than their fellows, it was by no means in such a proportion and it seems safe to assert that their salaries were at least 150% of what would have been attainable under similar conditions by the average engineer of equal length of practice. Mr. Stiles.

The writer heartily endorses the suggestion of Mr. Lundgren† that much valuable information might be obtained from a study of the salary lists of Government bureaus and large corporations, none of which seems to have any difficulty in obtaining plenty of qualified applicants for engineering positions at salaries much inferior to the Committee's averages. In the engineering department of one large railway, receiving hundreds of applications yearly, all college graduates without practical experience were informed that they would be required to begin as "track apprentices", at the same wages as common

* Continued from February, 1914, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., for February, 1914.

Mr. Stiles. labor. No particular effort was made to instruct such men as accepted this offer. They were assigned to section gangs apparently at random, and sometimes acquired valuable experience in picking up wrecks, relaying rail, ballasting, etc.; in other cases they spent months in sweeping stations, cutting grass, picking up scrap, and similar tasks. A large proportion quit in disgust and at least 75% of the "graduates", expressed themselves as considering their apprenticeship as time lost. On the completion of the "course", having at least qualified for gameness, they were employed as assistants on surveying parties at initial salaries of $50 per month and field expenses, rising gradually from "stake artists" to levelmen and transitmen. It was the announced policy of the department to promote those who made good at the rate of $5 per month after each half year's service, and to employ experienced men from outside sources at equivalent rates, so that, after 5 years' service, an assistant engineer would receive $100 per month. About 75% of them never got any higher, and finally resigned. Division engineers were paid from $150 to $200 per month, or perhaps more in rare instances, and about 5% of the entire engineering force received salaries in excess of this rate, after from 10 to 20 years of experience. Many young engineers take their first few jobs almost entirely regardless of remuneration, and on account of the valuable experience which they expect to acquire. It is much to be regretted that in many instances such experience fails to materialize, and they find themselves in the position of having purposelessly cheapened their own services and those of their fellow workers.

The advertisements frequently seen in the technical press in which persons claiming to be engineers of some experience make such statements as, "salary no object", "will accept anything", "desire to obtain work without resorting to pick and shovel", etc., reveal the principal reason for the low salaries paid to engineers in subordinate positions. The attitude indicated by these advertisers should be considered as highly unethical and steps should be taken by the Society to prevent its continuance. No union man out of work would endeavor to obtain new employment by agreeing to accept less than the standard wages of his trade, and an engineer of 10 years' practice who is unable to obtain a suitable position at a fair salary, should take up some other line of work for which he is better qualified, rather than lower the remuneration of the entire profession by continuing on engineering work at the wages of unskilled labor. It must be admitted that it would be a very difficult matter to effect the change of attitude necessary to reach the root of this evil, which would require that all engineers receiving $200 per month or less should be organized somewhat on union lines, a proceeding that few seem to consider practicable. As a preliminary to the definite discussion of any such plan, it is suggested that the Committee would do well to obtain some first-

Mr. Stiles.

hand information from the less fortunate elements of the profession by answering a number of current advertisements for positions or even by inserting advertisements of its own, obtaining detailed information from all applicants and tabulating the results.

The remarks of Mr. McCullough* in regard to the numerous engineering employment bureaus are of such importance as to merit much greater amplification. Theoretically, such institutions fill a legitimate field in investigating the past records of engineers and in furnishing competent men to employers on short notice. It is possible that there are individuals and corporations in need of engineering services who could obtain them in no other way so satisfactorily as through these agencies, but the writer's experience leads him to believe that *bona fide* cases of this kind must be comparatively rare. A new enterprise might secure a chief engineer from such a source but he, in turn, should have little or no difficulty in obtaining his staff from among his former professional associates and their acquaintances, reinforced, if need be, in the lower grades, by men obtained through advertisements or by recommended graduates of technical schools. Nevertheless, it appears that large numbers of men are constantly employed through agencies, chiefly on account of the fact that many employing engineers appear to have little time for, or interest in, this phase of their work, and often allow it to be handled and controlled by unscrupulous subordinates. The writer is inclined to believe that a considerable majority of the engineers employed through bureaus are, in one way or another, the victims of exploitation. The establishment of the proposed employment bureau by the Society and the pledging of all members to employ only men recommended by it, as suggested by Mr. McCullough, would certainly be of incalculable benefit to a great number of engineers.

As most registration fees are now made refundable in the event that no position is secured, the profit of a bureau depends entirely on placing men. The legitimate demand for such service being relatively limited, employees of bureaus are accustomed to increase their business by invoking the personal interest of the appointing power. This interest is usually secured by paying to the person influencing the appointment approximately one-third of the fee paid by the appointee to the bureau. Many men have accepted money from their appointees in this indirect manner, who would not think of blackmailing them directly. As far as known to the writer, there is no agency which does not grant commissions of this nature in return for services rendered by "insiders". Under such conditions, there is little prospect of efficiency in an engineering organization, every transaction being made with an eye to the fees obtainable rather than to the interests of either employer or employees. Each separation from the service means the creation

* *Proceedings*, Am. Soc. C. E., for February, 1914.

Mr. Stiles.

of a new vacancy; each appointment is the opportunity for another fee; every competent man employed tends to block future operations; every man of ordinary incompetency is an asset, as there is always valid reason for his discharge; and all promotion or other fair treatment of employees becomes false doctrine. The writer has never known any organization where these influences were in full control, but he has become acquainted with several where they were very potent.

In the case of one large company doing business in South America, considerable difficulty was encountered in getting a sufficient number of engineers experienced in tropical work, and, consequently, the salaries offered were relatively high. Yet applicants for positions who addressed themselves to the chief engineer's office were answered by form letter about as follows:

"We are in receipt of your application for a position with this company and will probably need an engineer of your qualifications in the near future. We have a contract with the ——— Engineering Agency for supplying us with all the men we may need and if you will make application through them there is little doubt that you will shortly secure the position you desire."

Application to this agency consisted chiefly in signing an agreement to pay a fee averaging about 30% of the first month's salary.

A chief draftsman in charge of about 30 men made a regular practice of replacing at least three every month at a personal profit of about $30. No matter how poor a draftsman might be, he was never discharged until he had entirely paid his fee, and, should he give satisfaction, he might remain almost a year, but was never promoted, higher positions being filled by new men paying larger fees. On the other hand, no applicant in person ever got a position but was referred to the ——— Bureau, which was thought to know of work elsewhere.

On a large construction job in the United States, about ten inspectors were employed through agencies, and averaged about 6 months' tenure of office. The contractors were doing very poor work, apparently in collusion with the higher officials of the company. If an inspector was honest and insisted on work approximately up to specifications, he was promptly discharged for some triviality. If he was guilty of accepting poor work, it was not discovered for some months and then the contractor was paid in full for such work and the chief saved his responsibility by immediately discharging the man found to have been incompetent and employing another, thus earning another fee.

The writer believes that the general condition of the Engineering Profession can be improved only by the gradual attainment of the following:

(1) Elimination of the great number of incompetents who now constitute such a large proportion of the lower grades;

(2) Standardization of the salaries of engineers, beginning with the subordinate positions and going as high as possible; Mr. Stiles.

(3) Prevention of the exploitation of engineers by the establishment of an employment bureau by the Society, to replace present private organizations in this field.

If the engineers in the higher grades will assist their juniors in this manner, they will find that in so doing they have bettered their own condition.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE DEPRECIATION OF PUBLIC UTILITY PROPERTIES AS AFFECTING THEIR VALUATION AND FAIR RETURN.
Discussion.*

By John W. Alvord, M. Am. Soc. C. E.†

John W. Alvord, M. Am. Soc. C. E. (by letter).—It is a matter of great interest that so many keen minds have devoted their close attention to the discussion of this paper. The subject is a rather abstruse one, but it is one that will have to be fully understood by engineers interested in appraisal work. It is, of course, impossible to review the many interesting points brought out in the discussion; only some general thoughts can be here set forth, suggested by the reasoning in certain typical discussions. Mr. Alvord.

The writer is interested to note, from Mr. Burns' discussion, the fact that the California, Idaho, and Missouri Utility Commission Laws require that sinking funds shall be established. To this information may be added the fact that the Indiana Utility Law, not only requires that such funds be established, but permits the reinvestment of such funds in extensions to the property, providing, however, that the value of such extensions shall not be added to the value of the plant as a basis of fair return or purchase.

Since this paper was written, the Society's Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities, has reported, and among its findings there is one (No. 14) in favor of deducting depreciation for the purpose of establishing fair rates. Later, the Committee issued an addition to the Progress Report, in which it amplifies methods of writing off depreciation, and concludes (page 18) "that any discussion

* Continued from February, 1914, *Proceedings*.

† Author's closure.

Mr. Alvord. of the subject which does not recognize the inter-relation between the method of providing for the depreciation and the method of valuing the property must necessarily be defective". Attention may be further directed to the fact that, since the paper was written, the Supreme Court, Appellate Division, First Department of the State of New York, in the Kings County Lighting Company *vs.* the Public Service Commission of the State of New York, First District, has upheld the Public Service Commission in its deduction of depreciation from reproduction-cost-new, reasoning out the matter in an opinion of several pages.

The writer is of the opinion, after a careful reading of the discussion and the Progress Report of the Committee on Valuation of December 1st, 1913, and its additional report, that what is needed at this time is a paper on fundamental principles of public utility valuation, and not papers dealing with details of rate-making and depreciation. The confusion of thought existing at this time appears to be caused by lack of understanding of the fundamental principles of utility valuation by a large number of engineers who are now interested in the question, and it will be hopeless to discuss details intelligently until a substantial agreement on such principles is had. As the fundamental principles of valuation work rest in the law and its interpretation by the Courts, many engineers apparently are not generally familiar with them. A brief effort will be made in this closure to clear up one or two of the more important ones which it is necessary to comprehend in order to read the discussions of this paper intelligently.

The writer would select for review, as typical, two of the discussions, one of which has apparently assented to the propositions contained in his paper, and the other of which has dissented therefrom. The first, by Mr. F. Lavis, points out (somewhat diffidently) what seems to him to be the difficulty with that class of valuers who dissent from the deduction of the depreciation from cost new.

To the writer's mind, Mr. Lavis strikes at the root of the matter. The difficulty clearly lies in the fact that the objectors do not take note of the fact that the appreciations of plants and property are included by the reproduction method, and must be considered in connection with the depreciations, and that, if the objectors would exclude depreciation, they must logically exclude the appreciation that the reproduction method usually introduces. The same arguments which one would use to exclude depreciation can be profitably used to show that questions of appreciation should not be entertained. This is fully stated in the paper.

It is a fair guess to say that failure to recognize this fundamental principle arises because some of the engineers now studying these questions for the first time have not yet had the opportunity to serve on actual appraisals, or, if so, they have not served on both sides of

the valuation problem, which is quite as important. This appears, to the writer at least, to explain the attitude of some of the more strenuous objectors to the theory here set forth. Mr. Alvord.

No one who has been in valuation work any length of time, and has served on both sides of the controversy, can fail to notice that there are three kinds of mental attitude among valuers:

1st. There are those who approach the subject from the standpoint of the protection of the rights of property and its conservation.

2d. There are those who approach the subject from the standpoint of the rights of the public and their interest in property devoted to public use.

3d. There are those, often recruited from the other two, who are influenced by the sincere desire to find the just and equitable relation between the public interest, on the one hand, and the private interest, on the other.

Now, in these matters we are, at the present time, really studying the third type of problem; that is, the just and proper relation between the public, on the one hand, and property, on the other, and not the protection of capital alone or the public alone.

We have found in the past, and undoubtedly shall continue to find in the future, that the public, in contested cases of valuation, is extremely keen, through its counsel and engineers, to ascertain and deduct every possible item of depreciation that can be taken away from a public utility property, and, on the other hand, that investors and owners, likewise through their counsel and engineers, are fully as keen to observe every fact and element which has appreciated their property. Both these two opposing tendencies are fundamentally sound, and, as usual, the truth lies in a careful conclusion which would allow to both all reasonable depreciation and obvious appreciation. When once the principle is grasped that both depreciation and appreciation must be taken into account, much of the trouble in dealing with this depreciation problem will have vanished. The Constitution of the United States gives the owner of a public utility property the right to have it valued "as of to-day," and as a "property", with all its accretions and growth in value, much of which may not be represented by any actual cash cost; and this right goes along with the right of the public not to pay a return on the full original cost of the property if it can be shown to have lessened in value in any way. This was stated clearly in the paper, but seems to have escaped the attention of some of those who have discussed the matter.

In the case of railroad properties, for instance, the appreciation of land values must certainly be a very large item, far outweighing any loss in value through minor depreciations of rails, cars, rolling stock, and the like. In the case of electric lighting properties, on the other hand, depreciations may often largely outweigh appreciations.

Mr. Alvord. The second discussion which the writer has selected as typical is that by Mr. Stuart K. Knox. His ideas have been set forth with great clearness and definiteness, and their amiable reasoning is so different from the unpersuasive tone adopted by Mr. Humphreys that one must be tempted, if possible, in the same spirit in which they are made, to note why one must differ from them.

Mr. Knox dissents from the writer's view on the second proposition, and introduces an interesting illustration (in amplification of the general statement in the paper), through which he seeks to show the absurdity of basing rates on depreciated property. He describes a water plant consisting solely of a pipe line, 10 miles long, costing $1 000 000, and having a life of 50 years, through which water, purchased at wholesale, is conveyed and sold at wholesale. A sinking fund of $6 550 is placed in the bank each year at 4% compound interest, which will amortize the entire line at the end of the assumed life. Mr. Knox reasons, as a first proposition, that if the sinking fund was kept intact as part of the property, the rates for service should be predicated on cost new, or, in other words, the accrued amount in the sinking fund should be permitted to earn at the same return as the rest of the property, and he concludes that this proposition, as set forth in the paper, appears to be "unassailable." The second proposition, however, he dissents from; that is, that if depreciation funds are detached or withdrawn from the property and used by the owner elsewhere in private gain, he cannot hope, as a matter of proper protection to the public, to receive rates which include a reserve fund which is not actually in hand, and so in hand that it is really part of the property. Mr. Knox proceeds to show that this proposition is not sound, by supposing that the owner of the pipe line does withdraw his sinking fund allowance and invests it elsewhere outside of the water property at the same rates of interest as it should earn as a sinking fund, and, further, supposing that the owner, in the tenth year of operation, becoming alarmed at the dwindling rates on his water transportation business, restores the fund, and christens it a "sinking fund", as part of the property.

This illustration is typical of a considerable number that have been introduced both in this discussion and in the report of the Committee on Valuation.

The difficulty of utilizing this kind of illustration is that not only is it quite unlike anything that happens in practically operated utilities, as has been set forth at length in the paper, but the further and more fundamental difficulty that it is not applicable to the problem here presented, because it is, pure and simple, the amortization of an original and actual cost. In valuation work, we are not considering the retracing of a past investment and its exact mathematical protec-

tion, at all, but we are considering present-day valuations made on a live and growing property in its entirety. Mr. Alvord.

So far as the writer's knowledge goes, it is not customary to treat the actual past cost as something from which depreciation should be deducted. It is true that the representatives of the public often make a plea for such a procedure, but Courts, commissions, or appraisal boards, with which the writer is familiar, have not decided that depreciation should be deducted from actual past cost for the purpose of making rates unless it can be shown that past cost is present value, nor does this paper even intimate the possibility of such a proposition.

What has been attempted to set forth is that in the reproduction method—that is, cost new as of to-day—if used for determining values, should have deducted from it the depreciation before using it for the computation of rates, if an adequate fund for depreciation is not in hand. A careful re-reading of the paper will make this clear. It is true that Mr. Grunsky calls it "past investment", but the writer only quotes Mr. Grunsky's exact language to make his own distinction, that it is "property now" that is valued, more conspicuous.

Now, Mr. Knox, in all his discussion, is evidently dealing with past cost, because in one place (page 216)* he says: "Assume that the reproduction-cost-new of the pipe is $1 000 000", and in the very next paragraph he says: "In the construction of this pipe line, the owner converted $1 000 000 of money into physical plant."

"Reproduction-cost-new" with him evidently means original cost, so far as his illustration and argument is concerned at least, for that idea runs through all of his succeeding reasoning on this illustration. On page 223* he says:

"On the other hand, if we neglect the fluctuations in the prices of labor and materials, reproduction-cost-new will remain an unvarying quantity as long as no additions are made to the property."

And, further:

"The reproduction-cost-new having once been ascertained may subsequently be kept up to date merely by adding the cost of extensions as these are made, and the 'adequate' annual depreciation reserve."

The writer has been actively engaged for the past 15 years in valuation work, and has never found yet that reproduction-cost-new would remain an unvarying quantity, even in plants which were not being extended. Nor can he understand how the value of a property, having once been ascertained, can subsequently be kept up by adding the cost of the extensions, for properties usually grow in value or decrease in value with the growth or decrease of the surrounding population and the general increase or decrease in opportunity to give service, and

* *Proceedings*, Am. Soc. C. E., for January, 1914.

Mr. Alvord. this entirely outside of and irrespective of the money actually put into them.

Mr. Knox must unquestionably realize, as any engineer would realize on second thought, that utility properties in actual life may appreciate or depreciate in value outside of their original cost, and that, if growth is found, it is conceded to be the property of the owner, by the Constitution of the United States, as interpreted by the Supreme Court in numerous cases.

Mr. Knox's discussion, therefore, has only been taken as typical of this neglect to take into account possible appreciation, but he is followed in this respect by no less than six out of the sixteen discussions. The writer will try to make this fundamental difference clear.

"Past cost" is commonly understood to indicate the actual monies originally expended for the upbuilding of a property. It is one of the elements to consider in reasoning from cost evidence to value, but the Courts generally hold that it is not a controlling element.

Past cost is seldom a good measure of value except in the case of quite recent expenditures. Past cost may have in it monies expended for obsolescent structures, but it contains none of the appreciations which commonly accrue to a property, and often accrue without actual expenditure. Because it has none of these appreciations, it is not proper to deduct from past cost the appreciations; and in actual practice it is not done unless some attempt is made to compute and add the appreciations as well.

Reproduction cost is the method of computing the cost to reproduce the "property" as of to-day, in a manner that is humanly possible, and at market prices for labor and material, and including the development of the business. Consequently, it contains many of the appreciations that have come to the property by reason of the growth of its environment. Not all of these appreciations have cost actual cash; some of them are natural accretions or earned and unearned increments, which have cost the owner nothing. From such an estimate should be taken the depreciations which the existing property has suffered by reason of its age, changing conditions, or use. After this is done, if the result, in connection with other lines of evidence, is found to be properly value, as distinguished from cost, then that value may be taken to estimate the fair return.

Now, all this is elementary, and in preparing the paper it was assumed that these fundamental principles were understood, but to guard fully against misconception at the very outset, the paper states that we are discussing only that line of evidence as to value known as the reproduction method, or cost-new-less-depreciation.

Mr. Wilgus speaks of a balance sheet showing an excess of assets over liabilities. This surely relates to past cost. Farther along he

Mr. Alvord.

speaks of an "impaired investment", showing that he has investment in mind and not reproduction-cost-new.

Mr. Willoughby says: "it is the ultimate finding of the accounting * * * that fixes the value of the utility," and though he immediately apprehends that we are limited to reproduction methods, yet he later speaks of such methods as though they were recent past cost, and in his illustration lapses into past cost completely again.

Mr. Boes clearly does not have in mind that we are discussing a "property", the value of which is fixed as of to-day.

Mr. Vensano argues entirely from past cost, and uses past cost in his illustration.

The Special Committee on Valuation of Public Utilities argues entirely from original cost, and, in its illustrations, appears to be constantly thinking about the proper protection of original cost.

Now, taking Mr. Forbes' example, and conceding his assumptions; that is, that original cost is "value" continuously through the life of the plant, the writer would promptly concede the correctness of his argument, that depreciation should not be deducted in any case for the purpose of making rates. There can be no rational dissent from such a proposition.

Practically, however, past cost is almost never present value. A thing may be worth what it cost, or it may be worth much more, or it may be worth much less; hence, past cost is very rarely accepted by the Courts as the best evidence of present value.

Even reproduction cost, less depreciation, is not always value. A plant may often be worth more or may be worth less than it would cost to reproduce it, and though some utility properties depreciate more rapidly than they appreciate, as a usual thing they appreciate much more rapidly than they depreciate, and both facts must be taken into account. In other words, original cost, plus the algebraic sum of all the changes in cost and theoretical depreciations, should equal reproduction cost, less depreciation, as of to-day, if it were humanly possible to make such an analysis correctly, but even so, we will not have arrived at value; other matters have to be considered as well.

In fixing fair rates, the Courts have said, over and over again, that it is the fair value (not cost) of the property used and useful for the public that must be taken into account.

It is interesting to note that most of the water-works engineers who have had experience in appraisal work have not misapprehended the fundamental proposition here set forth. Mr. Kiersted and Mr. Burns, particularly, have thrown a great many interesting side-lights on the main question.

To emphasize the matter further, it may not be inappropriate to revert to Mr. Humphreys' illustration of the hen (page 227* of Mr.

* *Proceedings*, Am. Soc. C. E., for January, 1914.

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Knox's discussion). Like a good many illustrations that do not illustrate, this one is misleading, in that it is not comparable with the actual conditions as we find them, for to make the illustration comparable with ordinary utility problems we should imagine that the hen lays more eggs as time goes by than she did originally, and is at the same time growing old.

What is the value of this hen to the consumer? Clearly more than it originally was, modified by the fact that she might soon die, and whatever the philosophy of the hennery farm manager, the Courts and commissions and practical appraisers would take both facts into account in fixing the price of eggs, especially if there were only one hen.

In conclusion, the writer is of the opinion that the trend of public utility regulation in the future is going to lean more and more to the requirement, in municipal utilities at least, that funds for depreciation, renewal, and contingency shall be largely, if not entirely, kept in hand as part of the property, subject to the control and supervision of the commissions. Such funds are not necessarily idle, as has been suggested, nor are they withdrawn from useful activity, but the public is undoubtedly much better protected, and the property is stronger and the owners' credit better when such funds are within public control and supervision.

It is a significant fact that in several cases, which have come under notice of late years, where new financing of public utilities, not under commission control, has been required, that the bankers have actually insisted on a replacement fund being created, and constantly maintained to a required amount, in order to make the property more sound and the securities issued more secure.

The case of the steam railroads presents a somewhat different aspect from the ordinary municipal utility. So much of the property of the steam roads is in land, which does not usually depreciate, and the remaining property depreciates so rapidly that, as pointed out in the Third Avenue Railroad case by Mr. Floy, it may be difficult and undesirable to create a special fund for depreciation, or find any method that would be more simple than that of maintenance, pure and simple. This, however, should not preclude the deduction of depreciation from reproduction-cost-new in cases of valuation where appreciations had been duly considered.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES EDMUND CHILDS, M. Am. Soc. C. E.*

DIED JULY 16TH, 1912.

James Edmund Childs was born at Neversink, N. Y., on July 11th, 1848, and began his railway work, in April, 1865, with the Engineering Corps engaged in the location and construction of the New York and Oswego Midland Railroad, now a part of the New York, Ontario and Western Railway.

During 1869 and 1870, he was Assistant Engineer on the Chicago and Michigan Lake Shore Railroad, now a part of the Pere Marquette Line. Later in 1870 he was Resident Engineer of the Buffalo, New York and Philadelphia Railroad, now a part of the Pennsylvania System, and in the following year, 1871, Division Engineer of the Rochester and State Line Railroad, now a part of the Buffalo, Rochester and Pittsburgh Railroad.

In 1873 Mr. Childs was engaged in the relocation of a division of the Wisconsin Central Railroad, as Division Engineer, and, in 1874 and 1875, as Assistant Engineer in charge of some improvements on the New York and Harlem Railroad, now a part of the New York Central System.

In 1876, Mr. Childs returned to the Rochester and State Line Railroad as Chief Engineer and Superintendent, and five years later, in 1881, came back, as General Superintendent, to the re-organized New York and Oswego Midland, on which as a boy, sixteen years before, he had begun his railway work.

He was also Assistant General Superintendent of the New York, West Shore and Buffalo Railroad during its construction and at the time it was opened for traffic in 1883.

In February, 1886, he was made General Manager and, in September, 1904, Vice-President and Director, of the New York, Ontario and Western. Mr. Childs was in the service of this company continuously for more than thirty years, with the exception of one year, 1889, when he was offered and accepted the position of Assistant General Manager of the Lake Shore and Michigan Southern Railway properties.

In the following year, however, he returned to the service of the New York, Ontario and Western, at the earnest request of its President

* Memoir prepared from notes by William A. Haven, M. Am. Soc. C. E., by Edward Canfield, M. Am. Soc. C. E.

and Board of Directors, in order to forward and complete the Ontario, Carbondale and Scranton Railway, then under construction from the main line to the City of Scranton, through the upper anthracite coal region. Mr. Childs remained in the service of this company as General Manager until the time of his death, filling also the positions of Vice-President and Director. By his energy and devotion to its interests and his knowledge of the property and territory which it served, he did more than any one else to promote its welfare and increase its traffic. His long service with the officers and employees, and the intimate and friendly character of his relations with them, rendered his death not only a loss to the company, but a personal sorrow to his associates.

In 1882, Mr. Childs was married to Laura Grant, a daughter of the late William H. Grant, M. Am. Soc. C. E., who survives him.

Mr. Childs was elected a Member of the American Society of Civil Engineers on December 4th, 1878.

JOSEPH POTTER COTTON, M. Am. Soc. C. E.*

Died December 13th, 1913.

Joseph Potter Cotton, a son of Isaac H. and Rhoda Lamont (Potter) Cotton, was born in Bowdoin, Me., on May 8th, 1837. He was descended from old New England stock. He received his education in the public schools and academies of his home district, and worked on the neighboring farms. Later, he taught in districts schools in Maine, New Jersey, and Pennsylvania.

While he was engaged in teaching at Easton, Pa., the Civil War broke out, and, in 1862, he served as Orderly Sergeant of a company of militia at that place. In 1863, he raised and took command of Company C, Forty-eighth Regiment, Pennsylvania Volunteers, which was mustered into the service of the United States.

In 1866, Captain Cotton began his engineering work as Rodman and Leveler on the survey for the Lake Superior and Lake St. Croix Railroad in Wisconsin. In September, 1867, he entered the Government service, under the late General G. K. Warren, Corps of Engineers, U. S. A., and was engaged as Assistant Engineer on surveys of various rivers and bridges in Minnesota, Mississippi, Wisconsin, and Ohio. In 1871, he went to Newport, R. I., and, under General Warren, had charge of the construction of the breakwater at Block Island. In 1872 and 1873, with the late E. S. Chesbrough, Past-

* Memoir prepared by the Secretary from information on file at the Society House.

President, Am. Soc. C. E., he made a survey of Newport, in connection with a plan for a new system of sewerage, which plan, slightly modified, was adopted and forms the basis of the present system. Captain Cotton continued in the service of the Government until 1883, being engaged on river, harbor, and fortification work. In 1882 and 1883, he served as Commissioner and Engineer on the Seekonk River Bridge, at Providence, R. I.

In 1883, he resigned to engage in the private practice of engineering. He made his home at Newport, and became actively interested in city affairs and public improvements. From 1876 to 1883, he served as a Member of the School Committee; he was also Overseer of the Poor for three years, and Street Commissioner in 1890 and 1891. He was one of the group of citizens who framed the present city charter, and, on its adoption, he became a member of the Representative Council, serving on several of its important committees. He continued as a member of the Council until his death.

Captain Cotton was also deeply interested in the social betterment of the city and its people. He was one of the founders of the Charity Organization in 1880, and from that time until his death was an active member of its Board of Reference. He assisted in establishing the Holly Tree Coffee Rooms, the Law and Order League, and the Industrial School, and was for some years a Trustee of the Newport Hospital. His greatest public service, and one in which he took the greatest interest and pride, was rendered in connection with the Newport Co-operative Society for Saving and Building, which he was largely instrumental in founding in 1888, and of which he was the first and only President. The growth, prosperity, and efficiency of this institution is owing more to him than to any other man.

On March 26th 1867, he was married to Miss Isabella Cole, who died in 1908. He is survived by two sons, Dr. Frederic J. Cotton, of Boston, Mass., and Joseph P. Cotton, Jr., of New York City.

Captain Cotton was actively engaged in work up to the very end of his long life. He died in his sleep on December 13th, 1914, without pain and without premonition of death.

A few days later, Rear-Admiral French E. Chadwick, U. S. N., retired, wrote of him:

"Our town feels that in the death of Captain Cotton it has lost a mainstay. This loss is not only in our being deprived of a wise counsellor, an upright, clear-thinking citizen working always for the good of his fellow-men, but we have lost an exemplar of character. To old men he was an example, in age, of activity in business and public life; to the younger he was a pattern, for he exemplified in very full degree what character in the large sense means."

Joseph Potter Cotton was elected a Member of the American Society of Civil Engineers on June 7th, 1876.

NED HERBERT JANVRIN, M. Am. Soc. C. E.*

Died July 17th, 1913.

Ned Herbert Janvrin was born at Somerville, Mass., on May 20th, 1871. He was the son of Hiram Gilmore and Catherine Marriott Plummer Janvrin. An ancestor was one Jean Janvrin of the Isle of Jersey, whose son, Captain John Janvrin, sailed from Lisbon, Portugal, in 1696, and settled in Portsmouth, N. H. John, a son of Captain John Janvrin, born in 1707, was graduated from Harvard College in 1728. William Janvrin, the great-great-grandfather of Ned Herbert Janvrin, married Abigail Adams, a niece of President John Adams. A maternal ancestor, Francis Plummer, emigrated from Wales, at the foot of the Snowden Mountains, in 1635, and with his wife and two sons settled on the bank of the Parker River in Newburyport. He was the first man to keep a tavern and operate a ferry in that part of Massachusetts.

Mr. Janvrin received his early education in the schools of Somerville, having been graduated from the High School in 1890. In the fall of that year he entered the Massachusetts Institute of Technology, from which he was graduated in 1894 with an S. B. degree. George F. Swain, Past-President, Am. Soc. C. E., writes of him that "he was a conscientious, faithful, and very capable student, lending his influence toward a proper standard of discipline and attainment, and he left the school with the respect and confidence of all his teachers."

Almost immediately after his graduation he entered the drafting-room of the Boston Bridge Works, serving there until May, 1895, under J. R. Worcester, M. Am. Soc. C. E., Chief Engineer, who was impressed with his brightness, quickness of comprehension, and accuracy.

From May to November, 1895, he was employed as a Traverseman with the United States Geological Survey on survey work then in progress in New York State. He then returned to the drafting-room of the Boston Bridge Works, remaining there until December, 1896. From that time until May, 1897, he was with Mr. Worcester, during which period the train-shed for the South Station, at Boston, was being designed. Mr. Janvrin did a great deal of responsible work in the way of calculating and checking, particularly in connection with the girder work of the midway floor. In this position he showed that he was developing as an all-around designer, retaining all he had learned by experience and earlier training.

From May, 1897, to July, 1899, he was employed in the Bridge and Construction Department of the Pennsylvania Steel Company, at Steel-

*Memoir prepared by Robert Ridgway, M. Am. Soc. C. E.

ton, Pa., as a Checker and as Engineer in charge of drafting on the designs of steel bridges and viaducts.

Between July, 1899, and March, 1900, Mr. Janvrin was Assistant Bridge Engineer for the Metropolitan Street Railway of Kansas City, Mo., designing several of the Company's bridges and viaducts. He then entered the service of Messrs. Waddell and Hedrick, of that place, remaining with them until August, 1900, during which time he was connected with the design and construction of the Kansas City Viaduct.

From September to December, 1900, he was again with Mr. Worcester, assisting in the design of the steelwork for the Boston Elevated Railway Company, and from January to March, 1901, he served with Norcross Brothers, Worcester, Mass., as a Draftsman on steel building construction.

In April, 1901, he went with the American Bridge Company, remaining with it until September 15th, 1905. Until August, 1901, he was a Draftsman in the Pencoyd Plant. From then until June, 1903, he was Draftsman and Assistant Engineer, respectively, in the Eastern Division drafting-room, then located at Pencoyd, Pa., and in the Eastern Division estimating-room, also at Pencoyd. From June, 1903, to the middle of September, 1905, he was employed as Assistant Engineer in the Erecting Department of the Pittsburgh Division. His work there was mainly in connection with contracts for which the Company had sublet the erection to others. Among such works may be mentioned the erection of the Louisville and Nashville Shops, at South Louisville, Ky., and the Norfolk and Western Bridge, at Portsmouth, Ohio. The notable qualities of his character, which were impressed on his superiors on this work, were his absolute honesty, his quiet, industrious habits, and his ability as a thoroughly competent field checker of detailed drawings.

From October, 1905, to April, 1906, Mr. Janvrin was with Herbert C. Keith, M. Am. Soc. C. E., Consulting Engineer, of New York City, as a designer on several heavy steel bridges for the New York, New Haven and Hartford Railroad. Here he showed a keen perception of the points involved in several special problems which were presented, and great ability in solving them.

On April 2d, 1906, he reported for duty as Assistant Engineer with the Board of Water Supply, City of New York, and remained in its service, engaged on designs and surveys for and the construction of the Catskill Water Supply System, until his death. His purpose in taking up this work was due largely to a desire to increase his knowledge in a branch of engineering different from that in which he had been so long engaged. During this period, as opportunities offered, he received several promotions in rank and pay, on the recommendations of his superiors, for efficient service. Until September, 1906, he

was assigned to the Reservoir Department, on general survey work for the Ashokan Reservoir, during which period he had charge of a stadia party on preliminary topographical surveys. He was then transferred to the Designing Division of Headquarters' Department, and in the following spring was placed in charge of the construction and testing of an experimental section of reinforced concrete pipe, 11 ft. in diameter and 210 ft. long. The experiment was made with a view of ascertaining whether this type of construction was feasible and desirable for some of the smaller siphons of the Catskill Aqueduct where the head to be provided for was not great. The pipe consisted of seven 30-ft. units, each unit differing from the others as to the mixture of concrete, the details of reinforcement, etc. Mr. Janvrin gave the details of this work his earnest and intelligent attention, and his report of it, and of the successful hydrostatic test which followed the construction, was of material assistance to the designing force and added much to the knowledge of the subject of reinforced concrete pipes. In December, 1907, he was transferred to the Northern Aqueduct Department and assigned to its Newburgh Division, in Orange and Ulster Counties, where he assisted in locating the cut and cover aqueduct on that Division of 15 miles. When the work was placed under contract, he was at first given charge of Contract No. 16, a section of aqueduct 2½ miles long, and, later, in June, 1909, of Contract No. 45, about 5 miles long. As the intensity of the work increased, Contract No. 45 was divided into two engineering sections, and on April 1st, 1910, Mr. Janvrin was given charge of the south section. A notable feature of his section was the building, in two places, of the large concrete aqueduct of 500 000 000 gal. daily capacity on a heavy foundation embankment of earth, where the surface of the ground fell below the elevation of the aqueduct invert. The larger of the two embankments was 900 ft. long, and its maximum height, from the surface of the ground to the bottom of the aqueduct structure, was about 18 ft. On the practical completion of this section, on May 1st, 1913, he was transferred to the Designing Division of Headquarters' Department, where he remained until his death. In this last period of his work, his thorough grounding in the principles of stresses and strains in structures and his complete knowledge of steel and concrete structures were of great assistance in establishing methods of design applicable to the many subsurface gate-chambers of the City Aqueduct tunnel.

While on a vacation he was stricken with spinal meningitis, from the effects of which he died at Boston, Mass., on July 17th, 1913. For nearly 20 years he had given his earnest attention to the active practice of his profession, and his death, in his forty-second year, ended a career of much promise.

A friend of Mr. Janvrin, who had known him from his boyhood, commenting on his characteristics and ideals, writes:

"The one thing that stands out is his single-mindedness. It has been my lot to know of no man whose life from beginning to end was so perfectly straightforward, whose character was more truly clear and clean, whose ideals were more high, but this singleness of mind stands out the most sharply. There was nothing complex in his make-up, and it seems to me in these days of mixed motives in the characters of the best of men oftentimes, that such perfect singleness of mind was as remarkable as it is rare. This same singleness extended to his intellectual processes. His really brilliant mind moved straight ahead always, without waywardness. In all his social relations, the same characteristic predominated. As a boy he was a generous, kindly companion and a helpful son. As a man he was upright and honorable with the world at large, staunch and loyal to his friends, unselfish and devoted toward all his family."

Another, who became associated with him in his work soon after his graduation from college, and who formed a friendship then which lasted as long as he lived, writes of him as

"A man of clean habits and high ambitions, of excellent ability, strictly honest and honorable in all his dealings, and true to his friends."

Those who knew Mr. Janvrin well recognize these traits, and will endorse the tributes to his character paid by these friends.

He was a Mason, a member of Temple Lodge 299, Kansas City, Mo., a member of the Technology Club of New York, and of the National Geographic Society. He was interested in outdoor sports and skilled in games.

He was married, on June 20th, 1907, to Miss Avis Genevieve Grimes, of Franconia, N. H., who survives him.

Mr. Janvrin was elected a Junior of the American Society of Civil Engineers, on October 5th, 1897; an Associate Member on June 5th, 1901; and a Member on April 4th, 1911.

DAVID NEILSON MELVIN, M. Am. Soc. C. E.*

DIED JANUARY 27TH, 1914.

David Neilson Melvin was born in Glasgow, Scotland, on July 21st, 1840. He was the son of David Melvin, of Paisley, Scotland, a successful card manufacturer at Oxford, England, and also a notable figure in the temperance movement in Great Britain.

* Memoir prepared by George A. Parker, Esq., Mech. Asst. to Supt., The American Linoleum Mfg. Co., Linoleumville, N. Y.

David Neilson Melvin received his early education at the Andersonian Institute in Glasgow, and, in 1855, was apprenticed to the firm of Crawhall and Campbell, Engineers, to study drafting and to work through the shops.

In 1861, on the completion of his apprenticeship, Mr. Melvin was employed as Draftsman and Mechanical Engineer by several firms in Glasgow and vicinity, particularly by Blake, Barclay and Company, for which company he designed and superintended the construction of machinery and fireproof buildings, for some of the largest sugar-refining mills in Great Britain. He also designed the machinery and buildings for sugar mills in the West Indies.

In July, 1863, he purchased an interest in a paper mill near Oxford, England, which he operated until the abolition of the British tariff on paper made the business unprofitable. In 1865, he went to Birmingham, England, as Assistant to Mr. Henry Lea, Civil Engineer, remaining in that position until April, 1867, when he came to the United States. Shortly after his arrival Mr. Melvin obtained a patent for an improved sectional safety steam boiler, and, later, for an automatic furnace door, and also for a high-pressure engine. He was also associated with Mr. T. A. Weston, the inventor of the differential chain pulley, in Buffalo, N. Y.

Later, he was employed as Engineer and Superintendent by the Rodgers Iron Manufacturing Company, of Muskegon, Mich., and A. F. Bartlett and Company, of East Saginaw, Mich., in the production of wood-working machinery and in the manufacture of steam engines. He also superintended the erection of some of the largest lumber mills in the Michigan lumber regions.

In February, 1873, The American Linoleum Manufacturing Company was organized, and Mr. Melvin was appointed its Engineer. With Mr. Frederick Walton, the inventor of linoleum, he designed and superintended the erection of the machinery and buildings of the Company's large plant at Linoleumville, on Staten Island, New York. On the completion of this work, he succeeded Mr. Walton as Superintendent.

In 1888, Mr. Melvin invented and patented the machinery for manufacturing an inlaid linoleum, and about 1900, he brought out a patent for wood inlaid. These goods are now being manufactured exclusively under his patents. He retained his position as Superintendent until his death, which occurred at Miami, Fla., on January 27th, 1914, after a lingering illness.

Mr. Melvin was one of the original members, and a Life Member, of the American Society of Mechanical Engineers. He was also a Member of the Richmond County Automobile Society.

David Neilson Melvin was elected a Member of the American Society of Civil Engineers, on July 3d, 1878.

PETER ALEXANDER PETERSON, M. Am. Soc. C. E.*

Died November 21st, 1913.

Peter Alexander Peterson was born in Niagara Falls, Ont., Canada, on November 8th, 1839. In July, 1859, after a thorough education, he was articled as a pupil in Surveying to Thomas C. Keefer, Past-President, Am. Soc. C. E., who, at that time, was Chief Engineer of the Hamilton Water-Works and of the Hamilton and Port Dover Railway. For two years, Mr. Peterson was engaged on these works, with headquarters at Hamilton, and from 1861 to 1863, on a survey for a canal from Georgian Bay to Toronto, being stationed at the latter place. In July, 1863, he obtained a license as an Ontario Land Surveyor, and during the next two years was employed on various engineering works under Mr. Keefer.

In May, 1865, Mr. Peterson left Mr. Keefer's employ to engage in the private practice of engineering, and during 1865 and 1866, he had charge of the reconstruction of three large dams on the Grand River, to replace those which had been carried away by floods caused by ice jams.

In the summer of 1867 he made surveys, plans, and estimates for the Petrolia Branch of the Great Western Railway, and, in the autumn of the same year, was appointed Resident Engineer of the Northern Division of the New York and Oswego Midland Railway. In March, 1868, he was appointed Resident Engineer of the Bathurst Division of the Intercolonial Railway, in New Brunswick, which was then being constructed.

Mr. Peterson remained with the Intercolonial Railway until September, 1872, when he resigned to accept the position of Chief Engineer of the Toronto Water-Works, the construction of which, it was estimated, would cost \$2 000 000, and included a filtering basin 3 000 ft. long, 10 000 ft. of conduit from basin to pumping well (4 500 ft. of which was a 36-in. flexible pipe laid across Toronto Harbor), pumping engines, reservoir, and more than 100 miles of distribution pipes.

In 1875, Mr. Peterson was appointed by the Quebec Government as Chief Engineer of the Montreal and Ottawa Section of the Quebec, Montreal, Ottawa and Occidental Railway, which, except for that portion between Terrebonne and Montreal, had been constructed between Quebec and Ottawa. The location of the unconstructed portion was under discussion and Mr. Peterson strongly urged the adoption of the direct line from a point near Berthier to Montreal *via* Bout de l'Isle, including a large bridge at the latter place. In July, 1878, however, the late Mr. Walter Shanly made a report to the Government of Quebec

* Memoir prepared by H. Irwin, Esq., Cons. Right-of-Way and Lease Agent, Canadian Pacific Railway Company, Montreal, Que., Canada.

in which he advocated the line finally adopted, namely, *via* Terrebonne, St. Vincent de Paul, and St. Martin's Junction, with two large bridges and two long, heavy grades up to and down from Mile End. Mr. Peterson's route was afterward adopted by the Canadian Northern Quebec Railway.

As an engineer, Mr. Peterson excelled in the location and building of the substructures of bridges, and as Chief Engineer of the Montreal and Ottawa Section of the Quebec, Montreal, Ottawa and Occidental Railway, he had charge of the construction of the Chaudière Bridge between Hull and Ottawa, having one span of 254 ft., one of 160 ft., ten of 150 ft., and one of 135 ft., and of several other bridges over the Rouge, North Nation, Lievres, and Gatineau Rivers.

In 1881, he was appointed Chief Engineer of the Atlantic and North West Railway Company, under the charter of which the Canadian Pacific Railway Company built the line from Mile End Station, near Montreal, to connect with the International Railway of Canada at Lennoxville. At the same time, but under another charter, this road was extended from the International Boundary, near Megantic, to Mattawamkeag, Me. While in this position, Mr. Peterson superintended the construction of the St. Lawrence Bridge at Caughnawaga, on this line, the piers of which, with the necessary lengthening, were strong enough to carry the heavy double-track steelwork which, by a strange coincidence, was completed only a month before his death. On this line were also several large steel bridges and trestles, including the bridge over the Richelieu River and high trestles at Ship Pond and Wilson Stream.

Mr. Peterson was also Chief Engineer of the Sault Ste. Marie Bridge and of the Mission River Bridge, in British Columbia, and of many other important works.

In 1890, he was appointed Chief Engineer of the Canadian Pacific Railway Company, which position he held until February, 1902, when he was obliged to resign on account of ill health. He then became the Consulting Engineer of the Company.

In August, 1903, he was appointed Chief Engineer of the Guelph and Goderich Railway then being constructed by the Canadian Pacific Railway Company, and held that position until 1908 when he was obliged to retire from active work owing to his failing health. He made his home in Montreal, Que., Canada, where he died on November 21st, 1913.

He was elected a Member of the Institution of Civil Engineers of Great Britain on December 1st, 1874. He was also a Charter Member of the Canadian Society of Civil Engineers, and served as Vice-President in 1889, 1892, and 1893, and as President in 1894.

Mr. Peterson had taken a leading part in railway construction in Canada during the latter part of the nineteenth century. He was

most conscientious in regard to the performance of his engineering work, and strict, but fair, in his dealings with others. He was very kind-hearted, and always willing to aid any of his staff, although he required from them close attention in their work and whole-hearted discharge of their duty to their employers. Being of rather nervous disposition, he was, at first, sometimes thought to be slightly abrupt in manner, but his unvarying courtesy soon removed that impression.

Peter Alexander Peterson was elected a Member of the American Society of Civil Engineers on January 5th, 1876. He served as a Director in 1892 and 1893, and as a Vice-President in 1896 and 1897.

WALLACE BERKLEY RIEGNER, M. Am. Soc. C. E.*

Died January 19th, 1914.

Wallace Berkley Riegner, the son of Aaron H. and Caroline S. Riegner, was born in Strawsburg, Franklin County, Pa., on January 27th, 1854. In his early boyhood his parents moved to Chambersburg, Pa., where he attended the public schools, from which, as well as from the Chambersburg Academy, he was graduated. He afterward entered Lafayette College from which he was graduated with high honors in June, 1877. While at Lafayette he received the Junior Mathematical Prize, the Senior Astronomical Prize, and was chosen to deliver the Honorary Philosophical Oration. He also represented his College in the Intercollegiate Mathematical Contest.

After his graduation, Mr. Riegner taught mechanical drawing in the public schools of Reading, Pa., for about one year. On October 27th, 1879, he entered the Engineering Department of the Schuylkill Canal where he remained until March 1st, 1880, when he went to Pottstown, Pa., as Assistant Engineer on the Philadelphia and Reading Railroad, remaining in this position until August 31st of the same year.

He then went South and from December, 1880, to December, 1881, was engaged on surveys and construction work, on the Elizabeth City and Norfolk Railroad. He afterward returned to the Engineering Department of the Schuylkill Canal, with headquarters at Reading, Pa.

In April, 1882, Mr. Riegner resigned his position with the Schuylkill Canal Company to enter the employ of the Philadelphia and Reading Railway Company. From April, 1882, to July, 1883, he was engaged as Division Engineer on the Shamokin, Sunbury and Lewisburg Division, and from July, 1883, to April, 1887, as Assistant Engineer on general field and office work and the design of railroad structures at the company's office in Philadelphia, Pa. In April, 1887, he

* Memoir prepared by William Hunter, M. Am. Soc. C. E.

was appointed Engineer of Bridges, which position he held at the time of his death which took place at Chambersburg, Pa., on January 19th, 1914.

Mr. Riegner was a Member of the American Society for Testing Materials, the Franklin Institute, and the Engineers' Club of Philadelphia.

He was an intelligent, energetic, and capable engineer, modest to a degree, with a strong grasp of details which enabled him to prosecute his work most successfully.

Mr. Riegner was elected a Member of the American Society of Civil Engineers on September 7th, 1904.

HENRY HELM CLAYTON, Jun. Am. Soc. C. E.*

Died April 8th, 1913.

Henry Helm Clayton, the son of Lillie Sale and John Benjamin Clayton, was born in Sappington, Mo., on February 8th, 1885. He received his education at the Kirkwood Public School and the Manual Training School at St. Louis, Mo., and was graduated from the latter in January, 1903.

Mr. Clayton began work as Rodman with the Terminal Railroad Association of St. Louis, where he remained until June, 1903, when he accepted a position on the International Mexican Railroad. Having won a scholarship in the Engineering Department of Washington University, he returned to the United States in September, 1903, to enter that University, and in June, 1907, he was graduated with the degree of B. S. in Civil Engineering.

He then went to Texas as Rodman on construction work for the Rock Island Railroad, but during the same year he accepted a position as Computer for the Board of Examination and Survey of the Mississippi River Commission. He remained in this position only a short time, returning to Texas to enter again the employ of the Rock Island Railroad Company, which position he retained until the work there was completed.

In January, 1908, Mr. Clayton formed a partnership for the private practice of engineering, with offices at Clayton and Kirkwood, Mo., which partnership was dissolved in December of the same year. In the meantime he had been appointed City Engineer of Wellston, Mo., but resigned this position to enter the service of the United Railways Company of St. Louis.

* Memoir prepared by John B. Clayton, Jr., Esq.

In May, 1909, Mr. Clayton joined the forces of the Missouri Pacific Railroad and continued in the employ of that company in various positions until his death on April 8th, 1913. In January, 1910, he was appointed Assistant Division Engineer on the Arkansas Division of the St. Louis, Iron Mountain and Southern Railroad. On June 1st, 1911, he was sent to Chester, Ill., as Assistant Division Engineer on the Illinois Division, where he remained until December 1st, 1912, when he returned to the Southern District.

Mr. Clayton was a man of irreproachable character and of strong personal magnetism, and was unusually energetic in the faithful execution of his work. Having a warm heart and a generous disposition, he was true and faithful as a friend. His make-up was that of a big man, and it is believed that he would have accomplished big things had his life been spared.

Mr. Clayton was a Thirty-second Degree Mason. He was elected a Junior of the American Society of Civil Engineers on December 3d, 1907.

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"**THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE.**" CHARLES H. LEE. (To be presented May 6th, 1914.)

"**CINDER CONCRETE FLOORS.**" GUY B. WAITE. (To be presented May 6th, 1914.)

"**CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION.**" W. C. HAMMATT. (To be presented May 20th, 1914.)

"**HUACAL DAM, SONORA, MEXICO.**" H. HAWGOOD. (To be presented May 20th, 1914.)

"**REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.**" HARRISON S. TAFT. (To be presented May 20th, 1914.)

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William P. Morse

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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(INSTITUTED 1852)

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MAY, 1914

Edited by the Secretary, under the direction of the Committee on Publications.

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

Telephone Number..................5913 Columbus.
Cable Address...................."Ceas, New York."

*Elected to fill the vacancy caused by the death of Emil Gerber, Director, on April 16th, 1914.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

April 15th, 1914.—The meeting was called to order at 8.30 P. M.; Vice-President J. Waldo Smith in the chair; Charles Warren Hunt, Secretary; and present, also, 162 members and 23 guests.

A paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled, "The Possibilities in Bridge Construction by the Use of High-Alloy Steels", was presented by the author, who illustrated his remarks with lantern slides. The paper was discussed by Messrs. N. Petinot, L. Moisseiff, Henry W. Hodge, F. W. Skinner, and the author. The Secretary reported that he had received communications on the subject from Messrs. M. J. Butler, Albert Lucius, and Charles Evan Fowler. These were not presented on account of lack of time.

H. de B. Parsons, M. Am. Soc. C. E., gave an illustrated description of the Plans for Main Drainage and Sewage Disposal for the City of New York.

The Secretary announced the following deaths:

James Lewis Frazier, of Louisville, Ky., elected Member, September 1st, 1880; died February 28th, 1914.

George Alfred Nelson, of Lowell, Mass., elected Member, April 4th, 1911; died June 3d, 1913.

Duncan Hugh Campbell, of Rio de Janeiro, Brazil, elected Associate Member, July 1st, 1909; died March 29th, 1914.

Philip Chapin Davis, of New York City, elected Associate Member, June 4th, 1913; died March 26th, 1914.

William Churchill Oastler, of New York City, elected Associate, March 31st, 1891; died March 31st, 1914.

Adjourned.

May 6th, 1914.—The meeting was called to order at 8.30 P. M.; President Hunter McDonald in the chair; Charles Warren Hunt, Secretary; and present, also, 144 members and 23 guests.

The minutes of the meetings of March 18th, and of April 1st and 2d, 1914, were approved as printed in *Proceedings* for April, 1914.

A paper by Guy B. Waite, M. Am. Soc. C. E., entitled "Cinder Concrete Floors", was presented by the author, and the subject was discussed by Messrs. W. B. Claflin, R. Montfort, C. F. Loweth, Charles C. Hurlbut, Oscar Lowinson, A. W. Buel, F. W. Skinner, and the author. The Secretary read discussions by Messrs. Arthur H. Diamant and J. R. Worcester.

A paper by Charles H. Lee, Assoc. M. Am. Soc. C. E., entitled "The Determination of Safe Yield of Underground Reservoirs of the Closed-Basin Type", was presented by the Secretary, and the subject was discussed by Messrs. James Owen and T. Kennard Thomson.

The Secretary announced the election of the following candidates on May 6th, 1914:

As Members

George Washington Biggs, Jr., Pittsburgh, Pa.
George Edward Campbell, Los Angeles, Cal.
Harry Vivian Francis, Darwin, Northern Territory, Australia
Fred Force Gordon, Rochester, N. Y.
James Ormerod Heyworth, Chicago, Ill.
George Forrest Maitland, Cheyenne, Wyo.
Robert Anderson Meeker, Plainfield, N. J.
Julius Kemble Monroe, Bruceton Mills, W. Va.
Frank Rhea, Washington, D. C.
William Little Seddon, Norfolk, Va.
Henry Thomas Shelley, Philadelphia, Pa.
Frederic Irving Winslow, Boston, Mass.
Eberhard John Wulff, Tarrytown, N. Y.

As Associate Members

John Flinn Ancona, Rochester, N. Y.
Horace Francis Anthony, Camanche, Iowa
Edward Adam Beck, Sewickley, Pa.
Charles Greenwood Benson, Washington, D. C.
Lucius Tullius Berthe, Charleston, Mo.
James Gibbons Browne, Navasota, Tex.
Howard Blaine Bushnell, Springfield, Ill.
Asa Clair Butterworth, Little Rock, Ark.
John Ross Chamberlin, Columbus, Ohio
William Gideon Closson, Brooklyn, N. Y.
Gaylord Church Cummin, Dayton, Ohio
Max Lee Cunningham, Oklahoma City, Okla.
Edward Myron Ellis, Minetto, N. Y.
Albert Theodore Goldbeck, Philadelphia, Pa.
Gilbert G Hall, South Bend, Wash.
Hipolit Mikolaj Hincz, Chicago, Ill.
Winfred Miller Kallasch, Tiltonville, Ohio
Frederick Liddell Macpherson, Edmonds, B. C., Canada
William Ray McCann, Culebra, Canal Zone, Panama
Floyd Francis McDowell, Yonkers, N. Y.
Leon Waddell Mashburn, Tunica, Miss.
Paul Bertolet Miller, Houston, Tex.
Arthur Roland Moore, Kelowna, B. C., Canada
Joseph Lincoln Murphy, Nelsonville, Ohio
Thomas Hartman Olds, Sorocaba, Brazil
Charles Wesley Petit, Oxnard, Cal.
Bion Harman Piepmeier, Springfield, Ill.
Frederick Henry Pond, Brooklyn, N. Y.
Arthur Rudolph Rhenisch, Oak Park, Ill.
James Rush Rhyne, Corning, Ark.
Herman Schove Schick, Manila, Philippine Islands
Nial Sherwood, Preston, Idaho
Frederick Pardon Sisson, Detroit, Mich.
William Wood Smith, Montreal, Que., Canada
Charles Henry Stevens, Philadelphia, Pa.
Franklin Stevens Storey, New York City
Ralph Watts Wardwell, Colorado Springs, Colo.
Ernst Victor Willard, St. Paul, Minn.
Guy Eric Woodward, Seattle, Wash.

As Associate

Nathan Abbott Bowers, Vancouver, B. C., Canada

As Juniors

Elmer Styner Blaine, Cape Girardeau, Mo.
Emanuel Leo Bolano, Albany, N. Y.
Edwin Gibson Bolger, Altoona, Pa.
Harry William Bolin, Berkeley, Cal.
Robert Hammond Boynton, Frankfort, Ind.
Paul Caldwell Campbell, Kansas City, Mo.
Sinclair Ernest Carpenter, Berkeley, Cal.
Lorenzo Todd Getty, Newcastle, Ont., Canada
Claude Franklin Hancock, Chassell, Mich.
Philip Zell Horton, Peoria, Ill.
Charles Christopher Kilby, New Haven, Conn.
Livingston Allaire Leeds, New York City
Donald Curtis May, Ann Arbor, Mich.
Arthur Herbert Morrison, Portland, Me.
Clifford Eaton Murray, Newark, N. J.
Quinces Robertus Nolan, Atlanta, Ga.
Adolph Joseph Post, Boston, Mass.
Walter Wesley Schuyler, Bocas del Toro, Panma
Lysle Enoch Spangler, Berkeley, Cal.
Walter Steinbruch, Brooklyn, N. Y.
Marion Jackson Verdery, Jr., Great Falls, Mont.
Robert Yule Walker, Belton, Tex.

The Secretary announced the transfer of the following candidates on May 6th, 1914:

From Associate Member to Member

Calvin Lewis Barton, New York City
McCrea Parker Blair, St. Boniface, Man., Canada
Elwyn Lorenzo Clarke, Sheridan, Wyo.
Harry Whiting Dennis, Los Angeles, Cal.
Charles John Eld, Jr., Little Rock, Ark.
Charles Rice Gow, West Roxbury, Mass.
Nathan Clifford Grover, Washington, D. C.
Clyde Leslie Huff, Athabasca, Alberta, Canada
Olaf Laurgaard, Laidlaw, Ore.
Boudinot Gage Leake, Fort Worth, Tex.
Morton Macartney, Spokane, Wash.
Stacy Steward Storer, Oklahoma City, Okla.
Wilkie Woodard, Los Angeles, Cal.

From Junior to Associate Member

Walter Benjamin Bushway, Boston, Mass.
J C Carpenter, St. Paul, Minn.

Gustavo Adolfo Dubois, Havana, Cuba
Chester Gordon Gillespie, Chicago, Ill.
Alvero Charles Gregson, Flushing, N. Y.
Arthur Raymond Holbrook, Brooklyn, N. Y.
John Charles Rathbun, Manila, Philippine Islands
Walter Farnsby Shaw, Barneveld, N. Y.
Vincent Reynolds Stirling, Zamboanga, Philippine Islands
John Leonard Vogel, Jersey City, N. J.

The Secretary announced the following deaths:

Howard Elmer Arthur, of Big Hollow, N. Y., elected Member November 8th, 1909; died April 19th, 1914.

Emil Gerber (*Director*), of Pittsburgh, Pa., elected Member, February 1st, 1888; died April 16th, 1914.

Thomas H. Johnson, of Pittsburgh, Pa., elected Member, September 5th, 1877; died April 16th, 1914.

Alfred Noble (*Past-President*), of New York City, elected Junior, September 2d, 1874; Member, April 3d, 1878; died April 19th, 1914.

Fred Thompson, of Washington, D. C., elected Member, October 1st, 1902; died April 22d, 1914.

Joseph Tintorer y Giberga, of Barcelona, Spain, elected Member, May 5th, 1880; died January 8th, 1914.

Lawrence Calvin Brink, of New York City, elected Associate Member, October 7th, 1908; died May 2d, 1914.

Aaron J. Zabriskie, of Jersey City, N. J., elected Junior, July 1st, 1885; died April 15th, 1914.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

May 6th, 1914.—The Board met at 3.15 P. M.; President McDonald in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bush, Edwards, Haskell, Hodge, Keefer, Leonard, Loweth, Montfort, Ockerson, Smith, Swain, Thomson, and Tuttle.

The following resolutions were adopted:

"*Resolved:* That $10 000 be allotted by the Board as the maximum sum available for the work of Special Committees during 1914, excluding unexpended balances left over from appropriations made previous to the present year."

"*Resolved:* That a Sub-Committee of the Board be appointed to consider the reports of Chairmen of Special Committees and to recommend to the Board such action as they consider desirable regarding the scope of the investigations proposed by and the expenditures of such Special Committees."

The President appointed as such Committee Messrs. Swain, Edwards, and Hodge.

The Secretary reported that $20 000 had been paid on the mortgage debt of the Society, which reduces the debt to $60 000.

The Constitution of the Louisiana Association of Members of the American Society of Civil Engineers was approved. The headquarters of this Association is in New Orleans.

Ballots for membership were canvassed, resulting in the election of 13 Members, 39 Associate Members, 1 Associate, 22 Juniors, and the transfer of 10 Juniors to the grade of Associate Member.

Thirteen Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

September 2d, 1914.—8.30 P. M.—A regular business meeting will be held, and two papers will be presented for discussion, as follows: "Some Principles Relating to the Administration of Streams", by Clarence T. Johnston, Assoc. M. Am. Soc. C. E.; and "The Construction of the Klondike Pipe Line", by W. W. Edwards, Assoc. M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

September 16th, 1914.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "The Constant-Angle Arch Dam", by Lars R. Jorgensen, Assoc. M. Am. Soc. C. E.; and "Subaqueous Highway Tunnels", by George Duncan Snyder, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-sixth Annual Convention of the Society will be held at Baltimore, Md., from June 2d to 5th, 1914, inclusive.

A circular giving full information in reference to the Convention was issued on April 24th, 1914.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general

books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendices* to the Annual Reports of the Board of Directors for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meeting)

February 20th, 1914.—The meeting was called to order at the Palace Hotel; President Snyder in the chair; E. T. Thurston, Jr., Secretary; and present, also, 80 members and guests.

President Snyder, in a brief inaugural address, reviewed the progress of the Association since its organization and the improved status of the engineer during the same period. He also submitted the following classification of the Members and Associate Members according to their specialties in 1907 and 1913, respectively:

	1907.	1913.
General construction	24%	26%
Building construction	15%	25%
Hydraulic	17%	18%
U. S. Government	8%	7%
Mining	13%	5%
Railway and traction	13%	9%
Municipal	10%	10%

The proposed Amendments to the Constitution of the American Society of Civil Engineers were discussed.

J. D. Galloway, M. Am. Soc. C. E., presented a paper on "Some Observations of an Engineer in Europe", which included an interesting account of many engineering features in Europe, particularly with reference to waterways, roads, and bridges, illustrating his remarks with stereopticon views.

Adjourned.

Colorado Association

The meeting of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Colorado Electric Club.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

April 11th, 1914.—The meeting was called to order at the Albany Hotel; President Ridgway in the chair; Roger W. Toll, Secretary; and present, also, 38 members and guests.

The minutes of the meeting of March 14th, 1914, were read and approved.

An invitation from the University of Colorado to hold the next meeting of the Association at the University, at Boulder, was presented and accepted unanimously, and the date of the meeting was set for May 16th, 1914.

F. H. Newell, M. Am. Soc. C. E., presented a paper on "The Work of the Reclamation Service", illustrating his remarks with stereopticon views. The subject was discussed by Messrs. A. P. Davis, J. C. Nagle, P. M. Norboe, W. D. Beers, A. J. Parshall, W. M. Reed, and others.

A vote of thanks was tendered Mr. Newell for his interesting paper.

Adjourned.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club.

At the meeting of the Association on December 29th, 1913, the new Chairman, John Ruddle, M. Am. Soc. C. E., was installed, and Messrs. Park A. Dallis and G. R. Solomon were appointed members of the Executive Committee. T. P. Branch, Assoc. M. Am. Soc. C. E., was elected Secretary.

Louisiana Association

At its meeting of May 6th, 1914, the Board of Direction considered and approved the proposed Constitution of the Louisiana Association of Members of the American Society of Civil Engineers.

Philadelphia Association

On December 22d, 1913, the Philadelphia Association of Members of the American Society of Civil Engineers was organized with the following officers: George S. Webster, President; Richard L. Humphrey and F. Herbert Snow, Vice-Presidents; John Sterling Deans, J. W. Ledoux, Edgar Marburg, and H. S. Smith, Directors; S. M. Swaab, Treasurer; and W. L. Stevenson, Secretary. The meetings of the Association will be held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

(Abstract of Minutes of Meeting)

April 6th, 1914.—The meeting was called to order at the Engineers' Club; President George S. Webster in the chair; W. L. Stevenson, Secretary; and present, also, 75 members and guests.

The by-laws which had been prepared by the Board of Direction, were submitted and approved.

Frederic P. Stearns, Past-President, Am. Soc. C. E., delivered an address on the "Valuation of Public Utilities," and the subject was discussed by Messrs. J. A. Emery, F. Herbert Snow, Thomas W. Hulme, E. D. Temple, E. Marburg, and John Sterling Deans.

Adjourned.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-

President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

(Abstract of Minutes of Meeting)

April 4th, 1914.—The meeting was called to order at the Commercial Club; President E. G. Hopson in the chair; Charles J. McGonigle, Secretary; and present, also, 11 members.

The minutes of the preceding meeting were read and approved.

A communication from Hunter McDonald, President, Am. Soc. C. E., asking for an expression of opinion on the advisability of the Association affiliating with local engineers who were not members of the Society, was read and discussed. On motion by Mr. Clarke, duly seconded, the following Resolution was adopted:

"That it is the sense of this meeting that the present organization answers the requirements of the American Society of Civil Engineers."

On motion, duly seconded, the Secretary was instructed to write to Mr. McDonald stating that the above Resolution had been adopted.

The Chairman or Secretary was ordered to communicate with prospective speakers and request them to prepare abstracts of their papers for publication, said abstracts, however, to be optional with the speakers.

The present policy of meeting once a month, as has been done since the formal organization of the Association, was approved by the meeting.

J. P. Newell, M. Am. Soc. C. E., presented a paper entitled "Depreciation as Applied in the Valuation of Public Utilities," and, on motion by Messrs. Clarke and Stubblefield, a copy of the paper was ordered to be forwarded to the Secretary of the Society for publication as part of the discussion on the Report of the Special Committee on the Valuation of Public Utilities. Mr. Newell was tendered a vote of thanks by the Association.

Adjourned.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year: Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, M. Am. Soc. C. E., 514 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in

Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

(Abstract of Minutes of Meeting)

April 8th, 1914.—The meeting was called to order; Vice-President Leeds in the chair; and W. K. Barnard, Secretary.

A paper by Dr. Ford A. Carpenter, in charge of the U. S. Weather Bureau Office at Los Angeles, entitled "The Excessive Rainfall of February, 1914," was presented by the author and generally discussed.

In view of the fact that there was an apparent paucity of funds for the maintenance of a sufficient number of observation stations in the vicinity of Los Angeles, a motion was presented and carried that a committee be appointed to investigate and, if possible, secure a fund for the purchase of additional observation instruments to be used under the supervision of the Weather Bureau Office, subject to the approval of the Chief of the Weather Bureau.

A motion was made and carried that the Board of Directors appoint a Committee to investigate the matter of representation of the American Society of Civil Engineers on the Building Commission of Los Angeles.

Adjourned.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, C. S. MacCalla; Vice-President, U. B. Hough; Second Vice-President, Morton Macartney; Secretary-Treasurer, A. D. Butler.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, Room 6, City Bank and Trust Company Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From April 2d to May 5th, 1914)

DONATIONS*

DESIGNING AND DETAILING OF SIMPLE STRUCTURES.

By Clyde T. Morris, M. Am. Soc. C. E. Third Edition, Revised and Reset. Cloth, 9¼ x 6¼ in., illus., 11 + 260 pp. New York and London, McGraw-Hill Book Company, Inc., 1914. $2.25.

The first edition of this book was issued in 1909, and in the preface to that edition, the author states that his aim has been to collect from the many and more exhaustive works on structural steel design, those parts which are applicable to simple structures and which can be taken up and taught in the limited time usually allotted to the subject in technical schools. He also desires to show, it is stated, by general cases and specific examples, how the simple laws of statics may be applied to the details of steel structures with the object of producing details which are in accord with the stresses they have to transmit. In this, the third, edition, and after four years' experience with the book in the classroom, the author has carefully revised the subject-matter, rearranged the chapters, changing the order of the presentation of the different topics, and added new material and illustrations. An entirely new chapter on highway bridges has been included, as well as a reprint of the Specifications for Steel Highway Bridges of the State Highway Department of Ohio. The Contents are: Designing and Estimating; Riveting; Mill Buildings; Plate Girder Bridges; Pin Connected Bridges; Details of Pin Connected Bridges; Highway Bridges; Manufacture and Erection; Appendix: General Specifications for Steel Highway Bridges, 1911, State Highway Department of Ohio; Index.

THE THEORY AND PRACTICE OF MECHANICS.

By S. E. Slocum. Cloth, 9½ x 6¼ in., illus., 42 + 442 pp. New York, Henry Holt and Company, 1913. $3.00.

It has been the author's aim, it is stated, to present the fundamental principles of mechanics in such a manner, in this book, as to emphasize their actual significance and relationship and, at the same time, to make them of interest to the average student of junior grade in colleges and universities, and in technical schools. In each article direct application to some practical problem is given with the explanation, and by thus properly combining theory and practice, the author states that the efficiency of instruction is greatly increased and mechanics also becomes a powerful instrument for co-ordinating mathematics and physics with technology. As this work is also intended as a reference book for the practising mechanical engineer, much has been added, it is said, which may be omitted in an elementary course of study. For a brief course the first three chapters are sufficient; for students in mechanical engineering Chapters IV and VII may be added, and for students in electrical engineering, Chapters V and VI. A feature of the book to which attention is called, is the collection of 420 practical problems most of which are original. The Contents are: Tables of Mathematical and Physical Constants; Kinematics; Fundamental Dynamical Principles; Statics; Friction and Lubrication; Kinetics of Particles; Kinetics of Rigid Bodies; Dynamics of Rotation; Index.

ECONOMICS OF INTERURBAN RAILWAYS

By Louis E. Fischer. Cloth, 7¾ x 5¼ in., 9 + 116 pp. New York and London, McGraw-Hill Book Company, Inc., 1914. $1.50.

In almost every community, the preface states, there are persons promoting, or encouraging the promotion of, an electric interurban railway. Many of such undertakings have been unprofitable, it is stated, because a proper scientific study of the fundamental principles governing their operating revenues and expenses, and cost of construction, has not been made, and the author, in this book, has given a résumé of actual economic results from the operation of existing electric interurban railways, in order to enable the layman or investor to comprehend more clearly the economic relations between operation and construction statistics and to discriminate between fundamentally good or bad investments in such projects. The Contents are: Inception and Development of Electric Traction; Classifications and Definitions;

* Unless otherwise specified, books in this list have been donated by the publishers.

Operating Revenue; Operating Expenses; Cost of Construction; Economic Relations, Operating Revenues, Operating Expenses, and Cost of Construction; Concluding Remarks; Index.

"THE ELECTRICIAN" ELECTRICAL TRADES' DIRECTORY AND HANDBOOK

For 1914. Thirty-Second Year. Cloth, 9¾ x 6¼ in., illus., 1752 + 106 pp. London, "The Electrician" Printing and Publishing Company, Limited, 1914. 15 shillings.

This Directory which is stated to be the most complete reference book published for the electrical and allied trades, was established in 1886, and is issued annually. In this the 1914 edition substantial and useful additions to the subject-matter have been made, it is said, and in view of the growing importance of several sections, the ground covered in previous issues has been greatly extended. Among the Contents are: Obituary notices of members of the electrical and allied branches who have died from January, 1913, to January, 1914; practical information relating to patent laws, designs, and trade marks, with a list of patents expiring in 1914; useful tables, statistics, and data relating to the various phases of electrical engineering; Acts of Parliament, Board of Trade rules, and laws relating to electric traction, lighting, power, etc., as well as British, Colonial, and foreign codes governing the application of electricity in mining; laws, codes, conventions, etc., relating generally to telegraphs, telephones, and wireless telegraphy; information and names of officers of electrical companies, and constitutions and officials, etc., of the various electrical engineering associations of the world; British universities, colleges, technical schools, etc., with professors, at which the study of electricity is a feature of the curriculum; a world directory of the professions and trades connected with electricity and its application, arranged alphabetically and classified by trades and professions; and a biographical section containing short sketches of the careers of many of the leading men connected with the electrical profession; indexes; etc., etc.

AMERICAN RAILROAD ECONOMICS:

A Text-Book for Investors and Students. By A. M. Sakolski. Cloth, 7¾ x 5¼ in., 12 + 295 pp. New York, The Macmillan Company, 1913. $1.25.

The Introduction states that the economic importance of American railroads and the participation of the people as individual investors in their rapid growth and development have created a demand for the proper understanding of railroad activities and operating results. Hitherto, it is said, statistical data of this sort have been compiled by professional analysts and railroad statisticians and the aim has been to gauge railroad activities by the use of rigid standards and definite mathematical formulas. The author, however, states that his purpose throughout this book has been a critical examination of facts and figures derived from railroad reports and other publications in order to assist in a correct judgment of railroad activities and operating results, without laying down any rules or maxims, and to give to each class of railroad data an underlying purpose, compiling, classifying, and interpreting each class to accord with such purpose. After preliminary chapters devoted to railroad rates, securities, and descriptions of the important railroad systems of the world, he has classified his subject-matter, therefore, into (1) data relating to the character of transportation problems; (2) data measuring efficiency and economy of operation; (3) data measuring revenues, expenses, and net earnings; and (4) data measuring the capital investment in relation to the corporate resources and liabilities. The Contents are: Railroad Rates; Railroad Securities; Railroad Systems of the United States; Economics of Railroad Construction; Physical Factors in Economic Operation: Way and Structures; Physical Factors in Economic Operation: Railroad Rolling Equipment; Traffic Statistics; Interstate Commerce Commission's System of Railroad Accounts; The Income Account; Net Income and Its Distribution; The General Balance Sheet; Railroad Capitalization; Index.

THE ENGINEERING CATALOGUES OF POWER-PLANT EQUIPMENT

For the Year 1913, Indexed by Firms and Products. "The Specification Digest" for Use in Drawing Specifications, and Making Purchases of Power-Plant Equipment. Compiled Annually by The Engineering Magazine Company. One-half Roan, 12½ x 9½ in., illus., 469 pp. New York, The Engineering Magazine Co., 1913.

This work is devoted, it is stated, to the field of power-plant installation, management, and superintendence, and brings into a single volume all material relating

to these particular subjects in such a manner that any item pertaining thereto may be located immediately, either by its general mechanical classification, its trade name, the name of the manufacturer, or the place of its production. The subject-matter is divided into three classes: (1) The Specification Digest which is a series of reminders of questions covering every point which the manufacturer or purchaser must answer fully and correctly in writing a specification or estimating for power-plant equipment. Following this part is a set of Standard Specifications for Horizontal Return Tubular Boilers adopted by the National Association of Tubular Boiler Manufacturers. (2) An Index to Products, under which is given alphabetically, the name of the manufacturer supplying the product, trade names, and page numbers indicating the page where descriptive and detailed information concerning the particular product may be found. (3) An Alphabetical List of the leading manufacturers and builders of power-plant equipment, whose catalogues are included in this book.

THEORY OF THE IRREDUCIBLE CASES OF EQUATIONS

And Its Application in Algebra, Geometry, and Trigonometry. By Charles Edgar White. Part II. Cloth, $9\frac{1}{2} \times 6\frac{1}{4}$ in., illus., 5 + 90 pp. Lancaster, Pa., Published by the Author, 1913.

Many general methods have been published, the author states, for the solution of cubics and biquadratics, etc., but, except in particular cases, they all involve imaginary quantities. The only two methods for the evaluation of the irreducible case of Cardan's formula that have been published are said to be Leibnitz's method by series and Eytelwein's and Könitzer's method by trigonometry. The author states that he has found, and has given in this book, an algebraic reduction of Cardan's formula by which the expression for the roots can be evaluated for any case of cubic equations for any number of decimal places. He states that many of the solutions, methods, and constructions included, are new mathematical literature, and that the solutions of equations and methods of computations given in Chapter II will be found to have a practical value in that by the formulas given trigonometric problems can be solved without the use of tables. Problems are given at the end of each chapter, and in these, it is said, may be found much that might have been included in the theoretical part of the subject-matter. The Contents are: Trigonometric Solutions of Cubics and Trigonometric Derivations of Algebraic Formulas; Approximate Solutions of Irreducible Equations and Methods of Computation; The Famous Problems in Elementary Geometry; The Inscription of Polygons; Constructions with Ruler and Compasses.

HANDY TABLES FOR COMPUTING THE COST OF TILE DRAINS.

Compiled by J. L. Parsons. Paper, 10 x 7 in., 9 pp. Humboldt, Iowa, The Author.

The first of these pages is devoted to Rules for finding average depths of trenches and for using the cost tables which are given. The following pages contain tables for finding the cost of trenching per 100 ft. at different prices per linear rod; the cost of trenching per 100 ft. at 1 cent per inch in depth per rod in length; cost of trenching per 100 ft. for different widths and depths at 30 cents per cubic yard; and for the reduction of decimals of a foot to inches.

A TEXTBOOK OF PURE MECHANISM.

By Frederick H. Sibley. Cloth, $9\frac{1}{2} \times 6\frac{1}{4}$ in., illus., 9 + 285 pp. New York, Henry Holt and Company, 1914. $3.00.

The preface states that no attempt has been made in this book to discuss a large number of mechanical appliances and machines, even of those in common use, the object being to select representative examples which best illustrate the geometry of machinery and to state them as briefly as possible. The classification used by the author is based, it is said, on the method of transmitting motion, which it is thought leads to a somewhat less complex treatment of the subject than that based on the method of making contact and used in several well-known textbooks. Certain fundamental rules for transmitting motion, which are stated as true of all contact motions, are given in Chapter III, and examples in subsequent parts of the text are constantly referred back to these rules. The method of treating cams and spiral gears is said to be original as far as the author knows. At the end of each chapter problems relating to the subject-matter contained in that chapter, are given. Much of the material for the text has been compiled from existing works on the subject and is said to represent little that is experimental. The Chapter headings are: Definitions; Graphical Analysis of Motion; Fundamental Rules in

Transmitting Motion; Link Connectors; Intermittent Motion by Linkwork; Wrapping Connectors; Trains of Mechanism; Transmitting Motion by Pure Rolling; Rolling and Sliding Contact; Direct Contact, Rolling and Sliding Motion, Spur Gears; Helical Gears; Index.

THE MINING WORLD INDEX OF CURRENT LITERATURE.

Vol. IV, Last Half Year, 1913. By George E. Sisley. Cloth, $9\frac{1}{4}$ x 6 in., 28 + 190 pp. Chicago, Mining World Company, 1914.

This Index, of which this is the fourth volume and covers the last six months of 1913, like previous ones, is said to cover the world's literature on mining, metallurgy, and kindred subjects. In it are classified all articles appearing in periodicals devoted to the subject, published in America, Europe, Africa, and Australia, together with publications of technical societies, Federal and State geological surveys and mining bureaus, as well as new books. The subject-matter is divided by classes, under which the entries are arranged alphabetically by subject and author, and includes title of article (which, if in foreign languages, is usually followed by translations or explanations in English), a brief digest where the title is vague, the name of the publication in which the article appeared or where abstracts of it may be found, the date and page number, the approximate number of words in the article, and the price. The Contents are: Publications Indexed; Metals and Metal Ores; Non-Metals; Geology and Mineralogy; Mines and Mining; Mill and Milling; Chemistry and Assaying; Metallurgy; Power and Machinery; Miscellaneous; Authors' Index; Subject Index.

STUDIES IN WATER SUPPLY.

By A. C. Houston. (Macmillan's Science Monographs.) Cloth, 9 x 6 in., illus., 12 + 203 pp. London, Macmillan and Co., Limited, 1913. $1.60.

The author, as Director of Water Examination of the Metropolitan Water Board of London, England, for eight years, has had, it is stated, exceptional opportunities to study and observe the system of water examination, researches, tests, methods of purification, etc., as carried out by that Board, and, in this book, he has brought together the results of his personal experiences and researches in relation to water supplies and methods of purifying them, which material previously had been scattered through a considerable number of reports and papers. In his chapter on Miscellaneous Information, he gives a list of the published Reports of the Metropolitan Water Board and a list of some of the books and reports on the Metropolitan Water Supply. The Chapter headings are: Sources of Water Supply; Researches Tending to Justify Rivers as Sources of Water Supply; The Question of Abstraction; Supplementary Processes of Water Purification; Sterilization Processes with Special Reference to the "Excess-Lime" Method; Storage in Relation to Purification; Water and Disease; The Financial Value of a Pure Water Supply; Bacteriological Routine Methods; Bacteriological Special Methods; Miscellaneous Information; Index.

POWER AND CONTROL OF THE GULF STREAM:

How It Regulates the Climates, Heat and Light of the World by Protecting the Warm North-Flowing Gulf Stream from the Onslaughts of the Ice-Cold South-Flowing Labrador Current. By Carroll Livingston Riker. Morocco, $7\frac{3}{4}$ x $5\frac{1}{2}$ in., illus., 102 pp. Brooklyn, N. Y., The Scientific Press, 1912. $2.00. (Donated by the Author.)

In this book, it is stated, the author sets forth a plan to intercept and turn the cold waters of the Labrador Current eastward at the Grand Banks of Newfoundland, by the construction of a jetty on the Banks, thereby securing to the Arctic regions some of the heat of the Gulf Stream and effecting climatic changes that would double the land values of the Northern Hemisphere. He describes the causes and effects of ocean currents and the equatorial and polar forces, as well as the location, construction, and cost of the proposed jetty. The Contents are: The Power of the Gulf Stream; The Grand Bank; The Jetty—Its Location, Construction and Effects; Manner of the Jetty Construction and Costs; Causes of Ocean Currents; Other Causes of Ocean Currents; Cause of the Gulf Stream; The Currents and Their Inclinations; The Central Sea; The Retroceding Strata; Effects of the Winds upon Ocean Currents; Influence of the Gulf Stream Upon the Inclinations of the Earth in Its Solar Orbit, and Some of Its Effects; Vertical Circulation of the Ocean and the Atmosphere; Index.

MACHINERY'S HANDBOOK FOR MACHINE-SHOP AND DRAFTING-ROOM:

A Reference Book on Machine Design and Shop Practice for the Mechanical Engineer, Draftsman, Toolmaker and Machinist. Second Edition. Morocco, $7\frac{1}{4}$ x $4\frac{3}{4}$ in., illus., 1 400 pp. New York, The Industrial Press, 1914. $5.00.

From time to time, every great trade, the preface states, should collect, systematize, and publish its established principles and working data, and, in this handbook, the compilers have sought to do this for the useful arts of machine design, construction, and operation. The subject-matter, it is stated, is the best of material which it has taken twenty years to collect, much that is published for the first time, and valuable information from hundreds of books and catalogues that have been searched for data to amplify or complete particular subjects. As stated in the secondary title, the book is intended for the use of the mechanical engineer, draftsman, toolmaker, and machinist, and although conditions, methods, and processes vary in machine-shop practice, it is hoped that the valuable information contained herein may prove useful to the whole mechanical field. A partial list of Contents is: Mathematical Tables; Logarithms; Mechanics; Strength of Materials; Riveting and Riveted Joints; Strength and Properties of Steel Wire and Wire Rope; Springs; Shafting; Friction; Bearings, Keys and Keyways; Clutches and Couplings; Friction Brakes; Cams and Cam Design; Gearing; Belts and Pulleys; Rope and Chain Transmission; Crane Chain and Hooks; Bolts, Nuts, and Machine Details; etc., etc.

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Das Entwerfen und Berechnen der Verbrennungskraftmaschinen und Kraftgas-Anlagen. Von Hugo Güldner. Dritte neubearbeitete und bedeutend erweiterte Auflage. Julius Springer, Berlin, 1914.

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Screw Propellers and Estimation of Power for Propulsion of Ships. By Charles W. Dyson. Vol. 1, Text; Vol. 2, Atlas. John Wiley & Sons, Inc., New York; Chapman & Hall, Limited, London, 1913.

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Investigating an Industry: A Scientific Diagnosis of the Diseases of Management. By William Kent. With an Introduction by Henry L. Gantt. John Wiley & Sons, Inc., New York; Chapman & Hall, Limited, London, 1914.

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REINSTATEMENTS

MEMBER	Date of Reinstatement.
MERSEREAU, CHARLES VERNON	April 1, 1914
JUNIOR	
WILDER, ALVIN DUMOND	Dec. 31, 1913

DEATHS

ARTHUR, HOWARD ELMER. Elected Member, November 8th, 1909; died April 18th, 1914.

BRINK, LAWRENCE CALVIN. Elected Associate Member, October 7th, 1908; died May 2d, 1914.

CAMPBELL, DUNCAN HUGH. Elected Associate Member, July 1st, 1909; died March 29th, 1914.

DAVIS, PHILIP CHAPIN. Elected Associate Member, June 4th, 1913; died March 26th, 1914.

FRAZIER, JAMES LEWIS. Elected Member, September 1st, 1880; died February 28th, 1914.

GERBER, EMIL. (*Director.*) Elected Member, February 1st, 1888; died April 16th, 1914.

JOHNSON, THOMAS H. Elected Member, September 5th, 1877; died April 16th, 1914.

NELSON, GEORGE ALFRED. Elected Member, April 4th, 1911; died June 3d, 1913.

NOBLE, ALFRED. (*Past-President.*) Elected Junior, September 2d, 1874; Member, April 3d, 1878; died April 19th, 1914.

OASTLER, WILLIAM CHURCHILL. Elected Associate, March 31st, 1891; died, March 31st, 1914.

THOMPSON, FRED. Elected Member, October 1st, 1902; died April 22d, 1914.

TINTORER Y GIBERGA, JOSEPH. Elected Member, May 5th, 1880; died January 8th, 1914.

ZABRISKIE, AARON J. Elected Junior, July 1st, 1885; died April 15th, 1914.

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(April 2d to May 5th, 1914)

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In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

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(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
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(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
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(33) *Le Génie Civil,* Paris, France, 1 fr.
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(35) *Nouvelles Annales de la Construction,* Paris, France.
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(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
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(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
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(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
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(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
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(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
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(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
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(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
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(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.

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Adiabatische Expansion des Wasserdampfes und die Expansionskurve der Dampfmaschinen.* Jar. Hybl. (53) Apr. 3.
Nordische Kalkwerke. (80) Apr. 4.
Kalkschachtöfen mit Generatorgasfeuerung.* Schmatolla. (80) Apr. 9.
Ueber den heutigen Stand der Wärm- und Glühöfen. (50) Serial beginning Apr. 9.
Neuere Selbstgreifer.* (50) Apr. 9.
Ueber die Wirtschaftlichkeit des Siemens-Martin-Verfahrens im Minettebezirk im Vergleich zum Thomas-Verfahren. N. Schock. (50) Apr. 23.

Metallurgical.

The Solidification of Metals.* Cecil H. Desch. (Paper read before the Inst. of Metals.) (11) Serial beginning Mar. 27.
Suggested Method of Standard Screen Tests. Lloyd Robey. (103) Mar. 28.
A New Continuous and Dustless Dryer for Sludge Material.* (105) Apr.
Some Notes on Stamp-Mills and Milling. Wm. H. Storms. (105) Apr.
Blast Furnace Blowing Apparatus.* J. E. Johnson, Jr. (105) Apr.
The Brier Hill Steel Company's New Works.* (20) Apr. 2.
Mastic Lining for Acid Tanks. (14) Apr. 4.
Blast-Furnace Bears and What They Teach Us. J. E. Stead. (Paper read before the Cleveland Institution of Engrs.) (22) Apr. 10.

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Metallurgical—(Continued).

Muntz Metal.* J. E. Stead and H. G. A. Stedman. (Abstract of paper read before the Inst. of Metals.) (47) Apr. 10.
Wrough Iron and Steel for Stamp Mill Parts.* Algernon Del Mar. (82) Apr. 11.
The Heat Treatment of Carbon Steel. Hugh P. Tiemann. (Paper read before the Technology Club of Syracuse.) (20) Apr. 16.
The Steel Hardening Process. R. H. Cunningham. (96) Apr. 16.
Electric Furnace for Smelting Various Ores.* Claude C. Whitmore. (82) Apr. 18.
Application of the Crushing Surface Diagram.* Arthur O. Gates. (16) Apr. 18.
A Self-Cleaning Screen for Pump Intake.* J. E. Johnson, Jr. (16) Apr. 18.
Ore Classification for Cyanidation.* Herbert A. Megraw. (16) Apr. 25.
A Reconstructed Russian Blast Furnace.* (20) Apr. 30.
Producing Steel in Electric Furnaces. Walter N. Crafts. (Paper read before the Cleveland Eng. Soc.) (20) Apr. 30.
Mild Steel and Its Treatment.* Albert Sauveur. (3) May.
Tube Mills *vs.* Conical Mills for Regrinding.* Julius I. Wile. (82) May 2.
Pyritic Smelting. George A. Guess. (Abstract of paper read before the Canadian Min. Inst.) (16) May 2.
Suspension Types of Ore Bins.* (16) May 2.
Umlaufende Gebläse für Giessereien und Hochofenbetriebe.* B. Weissenberg. (50) Mar. 26.
Die Gesetze des Uebergangs des Karbidsystemes in das Graphitsystem.* W. Guertler. (50) Serial beginning Mar. 26.

Military.

The Fried. Krupp Gruson Works, Magdeburg-Buckau.* (11) Apr. 24.
The Guns of Panama.* Charles M. Maigne. (46) May 2.

Mining.

Further Researches in the Microscopical Examination of Coal, Especially in Relation to Spontaneous Combustion.* James Lomax. (Paper read before the Manchester Geol. and Min. Soc.) (106) Vol. 46, Pt. 4.
Reineke Wireless Telephone for Mines. T. M. Winstanley Wallis. (106) Vol. 46, Pt. 4.
The Automatic Distribution of Stone-Dust by the Air-Current.* H. W. G. Halbaum. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 47, Pt. 1.
Notes on Gob-Fires and Blackdamp, etc.* John Morris. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 47, Pt. 1.
Stone-Dusting at Bentley Colliery: Report to the Doncaster Coal-Owners' (Gob-Fires) Committee. Robert Clive. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 47, Pt. 1.
New Interest-Tables for the Valuation of Mineral Properties. T. A. O'Donahue. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 47, Pt. 1.
Description of the Headings Driven from the Aldridge Collieries to Prove the Mines Over the Main Eastern Boundary Fault.* F. Bernard Clark. (Paper read before the South Staffordshire and Warwickshire Inst. of Min. Engrs.) (106) Vol. 47, Pt. 1.
Fuel Economy at Collieries by Means of Gas Power.* A. T. Cocking. (Paper read before the Midland Inst. of Min., Civ. and Mech. Engrs.) (106) Vol. 47, Pt. 1.
Rock-Drill Repair Costs.* C. K. Hitchcock, Jr. (6) Jan.
The Physiological Characteristics of Acetylene, with Respect to Its Use in Mining. E. E. Smith. (Paper read before the Inter. Acetylene Assoc.) (6) Jan.
The Valuation of Mines.* J. D. Kendall. (68) Mar. 21.
Hydraulicking on the Klamath River.* J. H. Theller. (103) Mar. 28.
The Development of Our Radium Bearing Ores.* L. O. Howard. (Paper read before the Utah Soc. of Engrs.) (1) Apr.
Bumps, Their Cause and Effect: A Description of Roof Movements in Mines at Coal Creek, B. C., That Have Resulted in Some Serious Accidents. John Shanks. (Paper read before the Canadian Min. Inst.) (45) Apr.
The Panel System in Ohio.* Wilbur Greeley Burroughs. (45) Apr.
German Coal-Dust Precautions.* Bergassessor Doctor Tornow. (Translated by Karl Kopp.) (45) Apr.
Selection of Induction Motors (for Coal Mines).* C. A. Tupper. (45) Apr.
Large Steam-Shovels for Stripping Coal Seams.* (13) Apr. 2.
Coaldust Experiments at Tirpentwys Colliery.* (57) Apr. 3.
Sinking Through Eleven Fathoms of Difficult Surface at Shettleston Colliery. J. Wilson. (Paper read before the Scottish Federated Inst. of Min. Students.) (22) Apr. 3.
Notes on the East Kent Coalfield.* E. Kilburn Scott. (22) Apr. 3.
Notes on Mesabi Range Mining Practice.* L. O. Kellogg. (16) Serial beginning Apr. 4.

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Mining—(Continued).

Fighting Dust with Dust, Powdered Stone to Prevent Coal Mine Explosions.* John B. C. Kershaw. (46) Apr. 4.
Method of Lining Shafts With Concrete.* Edward Morlae. (82) Apr. 4.
Calculation of Strike and Dip.* Theodore Simons. (16) Apr. 11.
Acetylene Lamps for Metal Mines. Frederick H. Morley. (103) Apr. 11.
Hydro-Electric Installations for Working the Mines of the Tekkah Mining Co. (Perak, Malay States.)* (26) Apr. 17.
The Oil Resources of the Empire. F. Mollwo Perkins. (29) Apr. 17.
Flame Phenomena Observed on Firing Permitted Explosives in the Mortar.* E. Lemaire. (From *Annales des Mines de Belgique.*) (57) Apr. 17.
A Modern German Colliery Plant; the Equipment of the Friedrich Heinrich Colliery.* P. Büssing. (From *Glückauf.*) (57) Apr. 24.
The Tangent System of Visual Signalling (for Mines).* (22) Apr. 24.
Automatic Controllers for Mining Work.* G. W. Humphry. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) Apr. 24.
Modern Methods in Placer Mining.* Carney Hartley. (82) Apr. 25.
The Use of Fuse and Detonators in Wet Blasting Operations.* Clarence Hall. (9) May.
Safeguarding Electricity in Mines. W. E. Freeman. (Paper read before the Kentucky Mining Inst.) (45) May.
Benzine Locomotives (for Mines).* A. J. Baijol. (Translated from *Annales des Mines de Belgique.*) (45) May.
Machine Mining in Room Pillars.* J. C. Edwards. (45) May.
Pond Creek Coal Co.* William Z. Price. (45) May.
Electric Mine Gas Detectors.* Sydney F. Walker. (45) May.
Bore-Hole System of Sand Filling.* W. A. Caldecott and O. P. Powell. (Abstract from *Journal,* Chemical, Met., and Min. Soc. of South Africa.) (16) May 2.
Quartz Mining in Colombia.* Ralph W. Perry. (16) Serial beginning May 2.
Report of Ray Con. Copper Co. of Arizona.* D. C. Jackling. (82) May 2.
Les Installations Minières dans le Bassin de Briey (Meurthe-et-Moselle).* E. Saladin. (32) Jan.
Signaux Electriques du Puits Sainte-Henriette aux Charbonnages du Bois Communal (Belgique).* (33) Mar. 21.

Miscellaneous.

Comparison of Estimated and Observed Values of Illumination.* W. C. Clinton. (Abstract of paper read before the Illuminating Eng. Soc.) (66) Mar. 31.
The Flow of Sand Through Orifices.* Ernest A. Hersam. (3) Apr.
Extinguishing of Fires in Oils and Volatile Liquids. Edw. A. Barrier. (55) Apr.
Requisites for Success in Engineering. Frederic H. Fay. (109) Apr.
Public Policy of Public Utility Corporations. H. S. Cooper. (111) Apr. 11.
High Tribunal Recognizes "Going Value" in Kings County Lighting Case. (14) Apr. 11; (13) Apr. 2.
Supervising the Execution of Large Percentage Contracts to Protect the Clients' Interests. G. G. Ommanney. (13) Apr. 30.
Artificial Daylight.* Herbert E. Ives. (3) May.
Special Representations in Specifications Control, General Cautionary Clauses, Recent Decision of the United States Supreme Court, Reversing the Court of Claims. William B. King. (14) May 2.
Notice Nécrologique sur Charles Tellier. G. A. Leroy. (32) Jan.
Les Lacs de Soude Naturelle.* P. Kestner. (32) Feb.
Le Système Taylor.* Ch. de Fréminville. (92) Mar.
Die Schallstärkemessung. R. Berger. (7) Mar. 14.

Municipal.

Methods of Paving Construction in Baltimore, Md.* Harry D. Williar, Assoc. M. Am. Soc. C. E. (60) Apr.
Brick Pavements: Their Maintenance and Repairs. F. F. Townsend. (60) Apr.
Municipal Improvement for 1914. (60) Apr.
Creosoted Yellow Pine Block Pavements in Dallas, Tex.* (60) Apr.
The Municipal Asphalt Paving Plant of Spokane, Wash.* (60) Apr.
Bituminous Pavements in Wilmington, Del.* (60) Apr.
Concrete Alley Pavements. John N. Edy. (60) Apr.
Methods of Making Surveys and Plans for Trunk Line Highways in Michigan. Harry L. Brightman. (Paper read before the Michigan Eng. Soc.) (86) Serial beginning Apr. 1.
Method and Cost of Constructing Slag Roads near Pascagoula, Miss. Charles E. Chidsey. (86) Apr. 1.
Roads of Brick or Concrete instead of Macadam for New York State Highways. (13) Apr. 2.
The Problem of Street Cleaning. S. Whinery. (From the *American City.*) (96) Apr. 2.

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Municipal—(Continued).

Have Bituminous Methods of Construction Solved the Modern Road Problem? W. H. Maxwell. (Paper read before the Institution of Mun. and County Engrs.) (104) Apr. 3.
Reverting Specification for Paving Bitumens. H. P. Pullar. (Abstract of paper read before the Mich. Eng. Soc.) (14) Apr. 4.
Kinks in Concrete-Road Construction. C. D. Franks. (Abstract of paper read before the Indiana Eng. Soc.) (14) Apr. 4.
Improvement of Chislett Street, Pittsburgh, Method of Supporting a Street over an Earth Slide by Using a Special Reinforced-Concrete Retaining Wall and Platform.* N. S. Sprague. (14) Apr. 4.
Methods and Costs of Constructing Gravel and Sand-Clay Roads in Perry County, Alabama. George C. Scales. (86) Apr. 8.
Some Good Roads, Their Construction and Maintenance. Robert C. Muir. (96) Serial beginning Apr. 9.
Methods and Cost of Laying 5 689 Sq. Yd. of Vitrified Brick Pavement at Carlisle, Pa.* John C. Hiteshew. (86) Apr. 15.
Design and Construction of Earth Roads in Iowa.* T. R. Agg. (13) Apr. 16.
Concrete Road Crossing a Flood Plain.* A. H. Hunter. (Abstract of paper read before the Ill. Soc. of Engrs. and Surv.) (14) Apr. 18.
Practical Instructions to State Aid Road Foremen in Wisconsin.* A. R. Hirst. (Abstract from *Bulletin No. 4,* Wisconsin State Highway Comm.) (86) Apr. 22.
Modern Road Work. H. P. Boulnois. (Paper read before the Roads Improvement Assoc. of Leicestershire, England.) (96) Apr. 23.
Sand-Clay *vs.* Macadam for Roads in the Southern States. (13) Apr. 23.
Recent Public Works at Chelmsford.* Percival T. Harrison. (Paper read before the Institution of Mun. and County Engrs.) (104) Apr. 24.
Summary of Street Traffic Conditions in a Number of Large Cities. (86) Apr. 29.
Protection of New Pavements against Destruction by Trenching, Macon, Ga. (13) Apr. 30.
New Specifications for Concrete Pavements, New York State Highway Commission.* (13) Apr. 30.
La Route Moderne. A. Sallé. (35) Serial beginning Apr.
Steinpflaster in Asphaltstrassen bei Verlegung von Gleisen. Günther. (39) Mar. 20.
Wie fördern wir praktisch das Siedlungswesen Gross-Berlins?* Fritz Beuster. (39) Apr.
Spiel- und Sportplatzanlage für Uelzen.* Victor Schmah. (39) Apr. 5.

Railroads.

Freight Train Handling. F. B. Farmer. (61) Jan. 20.
Heavy Track Scale on Buffalo, Rochester and Pittsburgh.* (15) Mar. 19.
The Electrification of the Usui-Toge Railway, Japan.* (From the A. E. G. *Journal.*) (23) Serial beginning Mar. 27.
4-6-0 Express Goods Engines, Caledonian Railway.* (21) Apr.
A Note on Staggered and Squared Rail Joints as Applied to Railway Tracks.* A. J. Beaton. (Paper read before the South African Soc. of Civ. Engrs.) (21) Apr.
A 2 000 I. H. P. Motor Locomotive.* (21) Apr.
Interesting Mikado Type Locomotive. W. H. Winterrowd. (25) Apr.
Northern Pacific Stock Car.* (25) Apr.
New Haven Steel Coach.* (25) Apr.
Brake Efficiency Tests on Steel and Iron Wheels. F. K. Vial. (25) Apr.
Grinding Wheels and Their Use. A. R. Davis. (25) Apr.
Improved Hanna Locomotive Stoker. (25) Apr.; (15) Apr. 3.
Panama Railroad Cross-Ties. (87) Apr.
Specifications for Scales.* M. H. Starr. (Paper read to the Am. Ry. Bridge and Bldg. Assoc.) (87) Apr.
British Locomotives in 1913.* J. F. Gairns. (88) Apr.
Electric Traction on the Simplon Railway.* Bruno Kilchenmann. (From *Bulletin* de l'Association Suisse des Electriciens.) (88) Apr.
Wooden Sleepers or Iron Sleepers. Ed. Lang. (From *Zeitung* des Vereins deutscher Eisenbahnverwaltung.) (88) Apr.
Congestion of Traffic. J. Hansen. (From *Zeitung* des Vereins deutscher Eisenbahnverwaltungen.) (88) Apr.
Electrification of Railroads in Switzerland.* E. Huber-Stockar. (65) Apr.
Light Signals.* C. O. Harrington, Jr. (3) Apr.
The Investigation of Railroad Accidents. Charles R. Vanneman. (36) Apr.
Preliminary Estimates in Connection with Railroad Work. Carl A. Gould. (36) Apr.
Construction of the Watauga & Yadkin Valley R. R.; with Details of Methods and Costs of Earth and Rock Excavation.* H. C. Landon. (86) Apr. 1.
Cost of Track Laying with a Track Machine. (86) Apr. 1.
The First 2 400-Volt Direct Current Railroad Switchboard.* (13) Apr. 2.

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Recent Improvements of the Boston & Maine Railroad in the Connecticut River Valley.* (13) Apr. 2.
The Rogers Pass Tunnel.* (13) Apr. 2; (96) Apr. 23.
Railway Shelter Stations of Unit Concrete Construction.* (13) Apr. 2.
Why Eastern Railways Need Higher Freight Rates. Daniel Willard. (Paper read before the Traffic Club of Pittsburgh.) (15) Apr. 3.
Pennsylvania's Improvements at North Philadelphia.* (15) Apr. 3.
The Rate Advance Hearings before the Interstate Commerce Comm. (15) Apr. 3.
Studies of Operation, the Pittsburgh & Lake Erie.* (23) Apr. 3.
Extension of the Charing Cross, Euston & Hampstead Railway.* (23) Apr. 3.
Railroad-Yard Lighting. (14) Apr. 4.
Shop Improvements of the Michigan Central R. R. at St. Thomas, Ont.* (18) Apr. 4.
Superheat.* E. J. Nicholson. (18) Apr. 4.
Stresses in the Plates of Cast-Iron Car Wheels.* Louis E. Endsley. (18) Apr. 4.
1 200-Volt High-Speed Passenger Locomotives on the Oakland, Antioch & Eastern Railway.* (17) Apr. 4.
The Rainy Lake Fill: an Episode in the Building of a Three-Thousand Mile Railroad.* Rex Croasdell. (19) Apr. 4.
Single Phase for the Rhaetian Railway.* (17) Apr. 4.
The 1 200-Volt D. C. Véberetsch Railway.* William C. Gyaros. (17) Apr. 4.
Flashlight Railway Signals. (13) Apr. 9.
The Position of the Pennsylvania Railroad. Samuel Rea. (Paper read before the Interstate Commerce Comm.) (15) Apr. 10.
New Dynamometer Car for the Baltimore & Ohio.* (15) Apr. 10.
The Possibility of Future Increases in Train Loads. Charles F. Speare. (15) Apr. 10.
The Construction of the Northwestern Pacific.* (15) Apr. 10.
The Railways' Attitude on Private Car Lines. (15) Apr. 10.
New Dock Facilities of the Hocking Valley Ry. at East Toledo, Ohio.* (18) Apr. 11.
Maintenance of Equipment Expense, Pennsylvania R. R. J. T. Wallis. (From Report to the Interstate Commerce Comm.) (18) Apr. 11.
Increase in Maintenance of Way Expenses on the P. R. R. J. G. Rodgers. (From Report to the Interstate Commerce Comm.) (18) Apr. 11.
Electric Traction on Lookout Mountain.* E. D. Reed. (17) Apr. 11.
New Double-Track Signals on the New York State Railways. (17) Apr. 11.
Converting a Tunnel into an Open Cut; Southern Pacific Ry.* George W. Wade. (13) Apr. 16.
Rail Breakages and Track Wave Motion in Cold Weather. (13) Apr. 16.
New Dining Cars, London & South-Western Railway.* (23) Apr. 17.
Water-Power for Railways in Sweden. (From Report of the Hydrographical Bureau of Stockholm.) (11) Apr. 17.
Operating Capacity of Single Track Divisions. W. M. Baxter. (15) Apr. 17.
Philadelphia & Reading 4-4-0 Type Locomotive.* (15) Apr. 17.
Extensive Reduction in Canadian Rates Ordered. (Board of Ry. Commrs. of Canada.) (15) Apr. 17.
Extensive Great Northern Snow Shed Construction.* (15) Apr. 17.
New Soo Line Freight Terminal in Chicago.* (18) Apr. 18.
Some Interesting Features of the Recent Derailment on the New York, New Haven & Hartford R. R.* (13) Apr. 23.
Comparative Study in Operation, Virginian and C. C. & O.* (15) Apr. 24.
Depreciation of Locomotives and Shop Equipment.* L. R. Pomeroy. (Abstract of paper read before the New England R. R. Club.) (15) Apr. 24.
Railways in China.* (12) Serial beginning Apr. 24.
Eight-Coupled Goods Engines for the Somerset and Dorset Joint Line.* (12) Apr. 24.
The New York, New Haven and Hartford Railway.* (12) Serial beginning Apr. 24.
New 4-6-2 Locomotive, Pennsylvania Railroad.* (23) Apr. 24.
Signalling Practice on the Eastern Bengal State Railway.* W. R. Bennett. (23) Apr. 24.
Heavy Freight Car Repair Facilities, L. S. & M. S. Ry., Ashtabula, Ohio.* (18) Apr. 25.
New Station for Panama R. R. at Panama.* (18) Apr. 25; (87) Apr.
Gasoline Locomotive for the Georgia Coast & Piedmont R. R.* (18) Apr. 25.
Illumination of Railway Signals. Thomas S. Stevens. (Abstract of paper read before the Illuminating Eng. Soc.) (13) Apr. 30.
Railway Track Scales and Weighing Methods.* Herbert T. Wade. (9) May.
Fuel Instruction Car on the Northern Pacific.* (15) May 1.
New Santa Fe Line near San Bernardino, Cal.* (15) May 1.
Rules Governing Weighing of Carload Freight. (15) May 1.
New Haven Improves Method of Electric Operation. William Arthur. (15) May 1.

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Christy Steel Freight Car Roof.* (15) May 1.
Triple Articulated Locomotive, Erie R. R.* (18) May 2.
New Freight and Engine Terminals, Air Line Junction, Ohio, L. S. & M. S. Ry.* (18) May 2.
Steel Passenger Car for the Canadian Pacific Ry.* (18) May 2.
American Type Locomotives for the Philadelphia & Reading Ry.* (18) May 2.
Reduction of Inductive Interference from the Power Lines of the New Haven Railroad. (17) May 2.
Minimizing Induction from Single-Phase Railway, New York, New Haven & Hartford R. R.* (27) May 2.
Appareils d'Attelage Automatique pour Wagons Primés au Concours de Paris (1912). (33) Mar. 28.
Double Croisement de Voies sur la Ligne du Great Southern Railway, à Buenos Ayres.* Ch. Béranger. (35) Apr.
Le Chemin de Fer Transafrican : son Tracé, les Méthodes de Construction et d'Exploitation d'après les Résultats des Dernières Missions.* R. Legouëz et R. Jullidière. (38) Apr.
L'Usine Electrique à Gaz Pauvre de la Compagnie du Chemin de Fer d'Orléans, à Tours.* H. Parodi. (33) Apr. 4.
L'Influence de la Retassure et de la Ségrégation sur la Résistance des Rails.* Ch. Dantin. (33) Apr. 11.
Neue Bauformen und Bauausführungen in Beton und Eisenbeton bei der württembergischen Staatseisenbahn-Verwaltung.* K. W. Schaechterle. (51) Serial beginning Sup. No. 7.
Wiederherstellung beschädigter Schraubenkuppelungen. Engelbrecht. (102) Mar. 15.
Zur Eisenbahn- und Schiffahrt-Frage in Kamerun.* (102) Apr. 1.
Die neue Bergbahn von Baden-Baden.* Frid. Rimmele. (40) Apr. 2.
Ueber das Rohrrinnen im Lokomotivkessel.* Oskar Prinz. (53) Apr. 10.
Lagerung feuergefährlicher Flüssigkeiten, Bauart Pintsch. (102) Apr. 15.
Schienenstühle auf kiefernen Schwellen.* C. Bräuning. (102) Apr. 15.
Befahren einer Langsamfahrstelle am Unterrichtsmodelle. Hans A. Martens. (102) Apr. 15.
Vierachsige Bahnpostwagen der schweiz. Postverwaltung.* (107) Apr. 18.

Railroads, Street.

The Buenos Ayres Subway.* (23) Mar. 20.
Advancement in the Street and Interurban Railway Industry.* Benjamin C. Tilton. (36) Apr.
The Design of Rolling Stock for Electric Railways.* H. E. O'Brien. (77) Apr. 1.
Concrete Pole Tests in Syracuse.* (17) Apr. 4.
East Side Tunnel of the Rhode Island Co., Providence.* Heaton R. Robertson. (13) Apr. 9.
Concrete-Mixing Car for Reinforced Decayed Poles.* A. J. Purinton. (17) Apr. 11.
The New Standard Grooved Girder Rail Section.* Martin Schreiber. (17) Apr. 11.
Report on Traffic Congestion in Fall River, Mass.* (Abstract of Report of D. C. and Wm. B. Jackson.) (17) Apr. 14.
Center-Entrance, End-Exit Cars for Pittsburgh.* (17) Apr. 11.
Passing of the Providence Counterweight Cable Road.* (13) Apr. 16.
The Passenger Transportation Problem. J. M. McElroy. (Report to the Manchester Tramways Dept.) (26) Apr. 24 : (104) Apr. 3.
The Boksburg (Transvaal) Railless Traction System.* R. Turnbull Mawdesley. (26) Apr. 24.
Sand-Handling by the Philadelphia Rapid Transit Company.* (17) Apr. 25.
Electrolysis Prevention in Edmonton, Alta.* W. T. Woodroofe. (17) Apr. 25.
Traffic Statistics in Pittsburgh.* (17) Apr. 25.
The East River Tunnels of the New Rapid Transit Lines in New York.* (13) Apr. 30.
Cantilever Canopies for Platform Shelters on the Chicago Elevated Railways.* (13) Apr. 30.
Chicago's Experience with Solid and Insert Manganese Special Track Work.* (17) May 2.
Overhead Problems, Spans, Brackets and Curves.* Charles Rufus Harte. (17) May 2.
Philadelphia Transit Construction ; Typical Designs for New Subway and Elevated Structures to Cost $57 578 000.* (14) May 2.
The Turbine Power Plant of the Louisville Railway Company.* W. O. Rogers. (64) Serial beginning May 5.
La Traction Electrique et le Système de Traction Auto-Régulateur (S. T. A. R.).* Paul Sauvage. (32) Feb.
Der Schienenreinigungswagen der städtischen Strassenbahn Zürich.* F. Largiadèr. (107) Apr. 18.

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Sanitation.

The Necessity of Ventilation. Meyer J. Sturm. (4) Mar.
Doncaster Refuse Destructor.* (104) Mar. 27.
The Calder Vale, Wakefield, Sewage Disposal Works. J. P. Wakeford. (Paper read before the Institution of Mun. and County Engrs.) (104) Mar. 27.
Hygiene and the Use of Ozone for Ventilation. Czaplewski. (Paper read before the Congress for Heating and Ventilating.) (105) Apr.
Design of the East Side Sewage Pumping Station, Hartford, Conn.* W. S. Brewer. (From paper read before the Connecticut Soc. of Civ. Engrs.) (86) Apr. 1.
Hypochlorite Water Disinfection and Typhoid Fever in Eight Cities. (13) Apr. 2.
Report Recommending Methods of Collection and Disposal of Municipal Refuse of Chicago. Irwin S. Osborn and John T. Fetherston. (Report made to the Chicago City Waste Comm.) (86) Apr. 8; (14) Apr. 11.
Operation of the Sewage Disposal Works at Atlanta, Ga. R. M. Clayton and W. A. Hansell, Jr. (Paper read before the Am. Assoc. for the Advancement of Science.) (86) Apr. 8.
Measuring Sewer Flow by a 26-ft. Sharp-Crested Weir.* (13) Apr. 9.
A Diagram for Solving McMath's Storm Sewer Formula.* R. W. Stewart. (13) Apr. 9.
The Bactericidal, Deodorizing and Physiological Effects of Ozone. F. V. Woolridge. (13) Apr. 9.
Removable Winter Inclosure for Sprinkling Sewage Filters, Gloversville, N. Y.* H. J. Hanmer. (13) Apr. 9.
Report on Collection and Disposal of Waste, Toronto. Geo. B. Wilson. (Report to the Board of Health of Toronto.) (96) Apr. 9.
New York Sewage Problem, Metropolitan Commission and Emscher Tanks. (104) Apr. 10.
A Discussion of the Present Status of Water Supply and Sewage Disposal Conditions in Chicago. (86) Apr. 15.
Typhoid and Paratyphoid along the Richelieu River. Theo. J. Lafreniere. (13) Apr. 16.
Reconstruction and Relief of the Rocky Branch Sewer; St. Louis, Mo.* W. W. Horner. (13) Apr. 16.
Transmission of Heat Through Building Materials.* Frank L. Busey. (64) Apr. 21.
Surveys and Plans for a Comprehensive Land Reclamation and Drainage Project.* (86) Apr. 22.
Removing and Washing Sand from Sewage Grit Chamber, New Bedford, Mass.* Walter N. Charles. (13) Apr. 23.
Operation of Imhoff Tanks. Charles Gilman Hyde. (14) Apr. 25.
Costs and Methods of Building Large Sewers.* H. R. Abbott. (Abstract of paper read before the Ill. Soc. of Engrs. and Surv.) (14) Apr. 25.
Method and Cost of Constructing an 18-in. Inverted Syphon for Sewer Crossing of Letort Spring, Carlisle, Pa. C. A. Bryan (86) Apr. 29.
Discussion and Data on the Correlation of Water-Borne and Some Other Preventable Diseases. C. M. Hilliard. (Paper read before the Indiana Sanitary and Water Supply Assoc.) (86) Apr. 29.
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PAPERS AND DISCUSSIONS

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Vol. XL. MAY, 1914. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

By Clarence T. Johnston, M. Am. Soc. C. E.

To be Presented September 2d, 1914.

Synopsis.

The laws generally applied in the United States in the administration of streams originated in the countries of Northern Europe. These countries developed under the feudal system, in contrast to Egypt, India, and China where, at a very early date, wise principles relating to the administration of streams were discovered and applied in practice.

Northern Italy is a striking example in so far as the development of principles relating to the administration of streams is concerned. Under the dominion of the Roman Empire, wise principles were applied. Spain brought other ideas of a beneficial character. The influence of the countries of Northern Europe, from which we borrow many of our laws and customs, was detrimental to the people of the Valley of the Po.

The doctrine of riparian rights is of feudal origin. It contains no positive principle, and it has outlived its usefulness. It can only be applied by the Courts, and the decisions of our tribunals of justice are

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

not in harmony as to the interpretations which should be given the doctrine according to character of uses and varying climatic conditions. We should not depart from a doctrine which has been followed for many years, until we have something better to suggest in its place.

The first principle that should be recognized is that of public ownership of streams, lakes, and other bodies of surface water. The recognition of this principle lays the foundation for an administration which, without great expense to the various water users, can protect all in proportion to the uses perfected, the character of such use, and with due consideration of all public interests.

Priority in time of use shall give the better right.

The right to use water gives the user no property interest in the water itself, and such right cannot be separated from the use, or transferred to other uses by such user. As long as the user has no personal control of the water supply, he has nothing with which he can speculate to the injury of others.

Preferred uses should be stated in order, in the law, and then protected in proportion to the value of each in preserving the lives of the people and promoting the general prosperity of any particular area.

Beneficial use should be the measure, the limit, and the basis of the right.

A declaration of abandonment should be made when the user fails to apply the water to some beneficial purpose within a reasonably short, specific time.

Even had these principles been recognized by the Courts, it would have been difficult for these tribunals to secure and judge the relative importance of the technical data that must be at hand before a just determination of rights may be made.

A compromise between Court and direct administrative control brings but little reform, because the responsibility is then divided and the defects of both systems are likely to be magnified.

The administration of streams lies within the field of the engineer. The best governed countries of the world, in this respect, have long since provided technical departments which administer this kind of public business in a very satisfactory way. The history of some of the countries of Europe is being repeated in America, and the engineer should be ready to accept the responsibility that, sooner or later, must come to him.

The manners and customs of the countries of Northern Europe have been introduced in the United States and accepted quite generally by the people. The climate and topography of the eastern half of the American Continent compare favorably with those of England, France, and Germany. It is natural, possibly, that the rules applied in the administration of streams, during the early history of America should be similar to the regulations enforced on the Thames, the Seine, and the Rhine. Where irrigation is essential to agriculture, we find an early interest manifested in problems relating to the diversion and use of water. The influences of the countries of Northern Europe in this particular would be more healthful if irrigation were practised there. It is probably unfortunate that the people of the United States have not been taught, by some early experience, that streams should be considered public property and one of the great natural resources.

In the older, irrigated districts of the world we find some good examples of wise laws relating to the administration of streams. China and India can teach us many lessons. The modern system adopted by Australia merits our study. Egypt, that land which has never enjoyed freedom, and has always been a victim of war, famine, and pestilence, administers its one river, the Nile, in a manner which can only call for praise from citizens of countries which boast of a higher type of civilization. Although the rulers of Egypt, both foreign and domestic, have robbed the people of their lands, have taxed them to the extreme limit, have compelled them to labor for the public without compensation, and by centuries of oppression have destroyed their initiative and broken their spirit, yet the Nile has always been public property. It is the great highway of commerce. It is the life of agriculture and the foundation of every other industry. The Nile is administered by public officers for the general benefit of the people. No private rights, which tend to threaten the prosperity of the people, are recognized or protected. The principles that have been accepted by Egypt are fundamental in their character, and they are applied uniformly throughout the extent of the Nile Valley. There is no conflict between farmers under the same canal, between communities depending on different canals, or between adjoining provinces. The principles that are essential are so simple that the native can understand and appreciate them. This immediately brings about a feeling of mutual confidence and respect between the water user and the officer in charge

of the local administration. Finally, there is no litigation in Egypt between users of water, whether they be individual farmers, corporations, or municipalities.

The laws of Imperial Rome, relating to the administration of streams, furnish examples which are superior to those we have introduced from the countries of Northern Europe. These laws were based on the idea of public ownership of streams. They recognized that water must be diverted from the natural channel of streams, in many cases, if beneficial uses are to be served and protected. They gave the builders of canals and other works the right to condemn private property. Those building improvements of this kind were obliged to respect the rights of others and to compensate those whose property was injured.

Northern Italy, like Egypt, has been a battle ground where the nations of the world have staged many important campaigns. Spain, France, and other nations have in turn dominated portions of the Valley of the Po. The influence of these invaders has been felt long after they surrendered political control. It would be assumed that the influence of Spain would not be detrimental in so far as the administration of streams was concerned. This was demonstrated. That country had gone through an experience that was profitable. The invasion of the Moors was beneficial to Spain in this respect, because these people from Northern Africa had been reared in districts where public control of the water supply was essential to life. The Spanish influence in Northern Italy, therefore, was salutary.

The influence of the people of France and Germany was detrimental. The countries of Northern Europe had developed under the feudal system. Because it was not necessary to divert large volumes of water from the streams to support the population, safe and sane principles were not evolved at home, and no apparent effort was made to discover and introduce rules that had been applied in countries which had experienced healthy, early development. The feudal countries always emphasized the importance of protecting private property for the benefit of the few owners thereof. The countries of Southern Europe and Northern Africa, on the other hand, recognized at an early date the necessity of public supervision of streams for the benefit of the many. When a portion of the Valley of the Po fell under the dominion of France, principles of feudal origin were intro-

duced. Concessions were made by the State and then by the Church. The fortunate individuals who received these concessions became owners of specific volumes of water, and speculation became general. The principles of the original Roman law were broken down here and there, and much confusion resulted. Litigation soon began, and many of these cases remained in the Courts in some form for several hundreds of years. Not until 1884 did Italy recover from this influence, in so far as its laws were concerned, and it will be many years before it recovers fully. Some plan must be formulated, and then carried into effect, whereby the old concession may be canceled. While Northern Italy was throwing off the customs it had inherited through early years of feudal association, some of our Western States were passing through a similar experience. The reforms that have taken place there and here are quite similar, yet neither country has profited by the experience of the other.

As the customs of Northern Europe, rather than those of other countries, were first introduced into the United States, we are somewhat concerned in their origin, the principles they embraced, and the method in which they were applied. As streams were of only passing importance to the feudal lords, and as private property was not held by the masses, it was probably apparent to them that a single rule might answer for all time. It was plain that some rule should be applied which would prevent the pollution of streams. As streams were of some importance to navigation, and as the owners of land bordering the watercourses desired stable conditions, in so far as discharge was concerned, it did not seem wise to encourage diversions from the rivers which would materially affect the flow thereof. The rule which gradually developed is approximately as follows: All persons owning lands abutting on a natural stream have the right to demand that the waters of that stream shall pass their lands undefiled in quality and undiminished in quantity. We call this the doctrine of riparian rights.

If this rule is carried into effect rigidly, water cannot be diverted from a stream in any way. The thirsty laborer who drinks from a convenient brook must diminish the "quantity" of water flowing therein. Streams must become more or less "defiled" with every contact with animal life. It was a theoretical rule, in the first place, this doctrine of riparian rights. As it had its birth in oppression and

despotism, so its application to the streams of our time has brought political and industrial slavery and injustice. The doctrine is a part of the unwritten law of England. Because it is theoretical in character and based on no well-defined working principles, it has furnished but little light to the Courts of our day.

The doctrine was introduced in the United States with the common law, and applied by our Courts, not by our law-makers. Its application has extended by judicial decree, so that its interpretations are only limited by the number of our Courts. It is seldom recognized and never defined by State or Federal statute. It was unjust to the Courts to thrust upon them the responsibility of protecting water users. When the law does not provide an administrative department for the purpose of transacting public business, those who would otherwise be protected must war among themselves and finally resort to tribunals of justice for relief.

We need no evidence, other than that contributed by the Courts themselves, to prove that our present methods of stream control, based on the doctrine of riparian rights, are not supported by principles that are sound. Every city, every water-power plant, every individual who resorts to the stream for domestic water supply, is violating the doctrine of riparian rights, yet this doctrine prevails almost alone east of the Missouri River. To be sure, the Courts hold that if the doctrine of riparian rights has been violated for a term of years the user may obtain a specific volume of water under what they call a prescriptive right.

Although this may give relief, in certain cases, yet it goes to demonstrate the weakness of the parent doctrine. It is an indication that the Courts recognize that some other rule must apply, if very obvious injustice is to be provided against.

The early decisions of the Supreme Court of Michigan hold that an owner of land abutting on a watercourse also owns the land of the bed of the channel to the "thread" of the stream. This term "thread" may be defined in theory, but presents some difficulties in practice. The Supreme Court of Michigan intimated, at first, that this was to be a general rule. It was to apply, and possibly has applied, to navigable streams as well as to those that are not navigable. Under this theory, the citizens of Detroit own the bed of the Detroit River to the "thread" of the stream. The same theory was stretched a little when

applied to the riparian lands bordering Lake St. Clair. It applied again to the St. Clair River, but when Lake Huron was reached the Court finally had to admit that the riparian owner had to limit his possessions at the meander survey of the lake. This would seem to show that a fundamental error had been made in the early decisions. The meander line should be considered the boundary between private and public property throughout. No private citizen can exercise exclusive control of the bed of a river without interfering with public interests. The doctrine of riparian rights, without modification, has but little to do with the protection of legitimate uses or the rights of the people at large. The Court, having no other rule to govern its acts, was compelled to construct its decisions with the timber at hand.

The writer has studied the application of the doctrine of riparian rights in countries of three continents. He has followed the history of its introduction into the United States, and watched its application as settlement has taken place, and as some kind of stream control has become necessary. He has found no two authorities who entertain the same views as to the theory of the doctrine, or as to its limitations in practice. If the doctrine has any value, if there are reasons why land owners along streams should have control of such streams, engineers, by this time, must have satisfied themselves as to the justification therefor. A discussion of the doctrine by engineers may be considered by some as an invasion of the field of the attorney. This is not true. It is not necessary to attempt to understand the intricacies of the theories that have been developed by the Courts. As the doctrine has been applied in the United States for more than 100 years, it should have demonstrated something as to its worth, and if it embraces a principle, this should be manifest to the layman. If the principles on which it is based are so obscure that they can only be understood and appreciated by the judiciary, then the doctrine should be abrogated, and something more tangible should be substituted in its place.

If the doctrine, such as it is, merits a stay of sentence, the engineer, who must always stand between the Court and the water user, should demand that its meaning be accurately and concisely defined, and its limitations be absolutely established.

The doctrine of riparian rights does intimate that the public (somewhat limited) is concerned in every stream. Court decisions, based

thereon, generally concur in the principle that the public owns the water, and only the use is acquired by private parties. The doctrine of riparian rights attaches the use to the lands owned by the riparian proprietor. A complete revolution is not necessary when we permit actual diversions of water from streams and place all rivers and their tributaries under the supervision of public officers. We simply pass from the application of a doctrine of negative principles to something positive and tangible. We may go back and study the application of the doctrine of riparian rights in other countries and satisfy ourselves as to the wisdom of affording it continued support here. When Great Britain enters new territory she does not apply the common law, to the detriment of her subjects. The doctrine of riparian rights does not apply in Egypt, South Africa, India, or Australia, as it does here. There, the Courts have never been compelled to make laws governing any resource.

Fortunately, we have some examples of better things, even in the United States. A few of our Western States have provided engineering administrations which, not only have custody of the records relating to the initiation of rights, but also determine the relative rights of all claimants. In order that such administration may do justice to private and public rights, principles are defined in the statutes that govern these departments. There, a land owner has no rights in a stream simply because his property is located along that stream. The great aim of the law is to encourage uses which develop the country, which establish homes, and, at the same time, preserve all public interests.

Before we discard a rule that has been applied for many years, we should have something better to substitute in its place. The administration of streams is very complex and difficult unless the few, wise, fundamental principles, discovered and enforced many years ago, are recognized. Though not assuming to refer to all principles that might apply, the writer will discuss a few of the more essential rules somewhat in the order of their importance. As has been intimated, the principle of public ownership of streams is the first essential to a just administration of the water supply.

a.—All streams, lakes, and other bodies of water, within the exterior boundaries of any country, should be, and always remain, public property. This means that the stream bed and the water flowing therein should be thus considered. There are many reasons why this prin-

ciple should be upheld. There is a physical reason, which is seldom considered. Streams are naturally of a public character, in so far as they are distinct from political lines. A stream, therefore, is a matter of general public concern. In addition, water cannot be privately owned. If Thothmes I had upheld the theory that private ownership of water is possible, and, to illustrate his ideas, had bottled a quantity and dedicated it to the generations of his family that followed him. the receptacle only would remain to those representing his house at the present time. Water is constantly moving. Like air, it is controlled by laws which were framed by a Higher Authority. We may enjoy the use, but not the possession.

As soon as streams, or any surface waters, become commodities for trade and speculation, the local public, responsible for such a condition, is openly defying natural laws and thereby imposing burdens on those who cannot protect themselves. No act of a despotic or careless government will work greater injury to its citizens than to permit, by decree or sanction of law, the private ownership of water. One might conceive of a government which would encourage a private monopoly of the air. It is at once apparent that this would be disastrous. Water falls in the same category, but because a profitable traffic can be carried on with apparent success, for a limited time, we are not shown, at once, the iniquity thereof. Though private parties have been able, frequently, to speculate in water, yet it is probable that the public has lost none of its inherent rights, and that both the buyer and seller have been deceived as to the real effect of the deal when consummated.

Public control of streams results in the determination of claims without great cost to water users. Where the Courts have jurisdiction, only the claimants who are financially able to maintain themselves in a controversy (that generally continues for years) can take an active part in the "adjudication" proceedings. Under the best administrative systems, the water user has no expense except that incurred in the payment of a nominal fee of two or three dollars. The public makes the surveys and measurements required, and collects all field data. The tabulation of rights is prepared by public officers, and hearings are provided where all users interested may protest against statements made as to the time water was used for beneficial purposes, and similar testimony. When the adjudication is accomplished, the final order is carried into effect by the same administration. All water users, con-

cerned in diversions from any stream and its tributaries, are brought together in the single proceeding, and no one, because of financial ability, or personal or political influence, has an advantage. High-class, public service of this kind is being performed in several of our Western States. The older administrations have had an experience of more than 20 years, and we can say now, with some assurance, that these are, in every way, a success. There, we find constitutional and statutory provisions which declare the waters of the States to be public, or the property of the State. These declarations relate to principles, and not to doctrines that have unlimited elasticity in interpretation.

b.—Priority in time of use shall give the better right. That is, the first user of the waters of a stream has a better right than any subsequent user, and the second user has a right superior to the third and those who follow, and so on. It is, perhaps, unnecessary for this principle to be discussed. It is absolutely necessary to make some such rule, particularly as between a large number of claimants concerned in a use of the same kind. It is manifest to one who gives the matter due consideration that, after the early users have made improvements and perhaps established homes, they should be protected as against later comers, who know what has taken place at the time of their appearance. This principle appeals with more force to an administrative officer, perhaps, than it does to one who has never participated in the practical work that falls to an engineer under such a system.

c.—The right to use water gives the user no property right in the water itself; such right belongs to the use and not to the user. This principle recognizes perpetual public ownership of water. The user is protected thereby, because he then belongs to a large class, all of whom are placed on the same basis, and no one person can impose on his neighbors. As long as the user has no personal control of the water supply, he has nothing with which he can speculate, to the injury of others. Although circumscribed by rigid restrictions, the legitimate user is protected, in so far as his use justifies protection, and, at the same time, he is given no special privileges.

Certain engineers framed a water law for one Western State some 20 years ago. They tried to have the measure specific in attaching the rights to use water for irrigation purposes to the land reclaimed, and to make it impossible for the owner of the land to dispose of such water right, except with the land. After some active campaigning

the bill was presented, and the framers were confident that this principle had been incorporated. They had a part in the administration of the law, and this principle was carried into effect. Within a short time an irrigator attempted to make a transfer of his water right separate from his land. Litigation began at once. This controversy in the Court was continued in an active manner for 10 or 12 years. At the end of that time the litigants had lost their farms, these having been consumed in Court costs and attorneys' fees. Finally, the Court decided that the law did not embrace the principle, and the transfer was thereby sanctioned. Those interested directly and indirectly in speculation in water joined in open hostility against any measure which had for its object the correction of the original law for the purpose of incorporating the principle therein. The friends of reform carried on a campaign for 6 years, and their efforts were finally rewarded by the enactment of adequate legislation.

To those who have observed problems of stream control from some distance, this principle may seem to be of but little importance. Wherever it has not been carried into effect extensive litigation has resulted and great injury has been done to water users who were not in a position to protect themselves. The argument made by those who oppose the principle is that any one who is damaged can secure redress in the Courts. This plan has but little to recommend it, because the weaker element is immediately eliminated from the contest. Further, the effect of a transfer, separate from the original use, is seldom apparent at the time of the transaction, and before those who are injured can appreciate the resulting damage and take steps to protect themselves, much money may have been spent in perfecting the new use. There is one exception to this general rule, however. It is often necessary to transfer rights to preferred uses. This should always be accomplished under public supervision. Should a city need water that had been formerly used for irrigation, power, or for other less important purposes, a procedure should be prescribed whereby a transfer of right to use (not a right to the water itself) may be made to the municipality in a public manner, so that all concerned may make their objections at the time, and, if injury results that cannot be foreseen, compensation can be made therefor under the direction of the administrative officers. Water cannot be maintained as public property when private parties sell water rights separate from the use to which they have been

dedicated, except under a public procedure and under the supervision and advice of public officers, so that, in carrying this last principle into effect, we are upholding another of equal importance.

d.—From the early records we are able to read it would seem that there has always smouldered in the breast of man, a feeling that all rights to use water cannot be classified in the same list. The first use that comes to mind relates to the domestic supply—water for drinking and household purposes. We must protect such uses, regardless of all other claims. It is plain that there should be preferred uses. There may be some room for debate as to the exact order in which these should be guaranteed protection. Climatic conditions, the nature of the water supply, and many other items enter as factors in such an undertaking, but an approximate estimate, which is of general application, can be made. No one will deny the necessity for protecting the individual supply for domestic purposes first. This is almost identical with municipal uses. Because many people, combined within the limits of a town, require more water than does any single claimant, the people of towns are often imposed on. Generally, under the doctrine of riparian rights, a farmer or other land owner is justified in taking water from a stream, adjoining his lands, for domestic purposes. As a rule, no effort is made to limit this use. We are compelled to suppose that it must be "reasonable". There seems to be no reason to presume that a single family, within the limits of a town, requires more water than does a farmer, living beyond its borders. Bearing in mind that the uses in both cases are equally high, and the only difference is one of local government, the question arises: When shall we deny the user within the town the same rights and privileges that are granted to the farmer? The Courts would probably justify a decision which discriminates between the two uses by the fact that the farmer is an owner of riparian lands and the city user is not. Assume that each user within the limits of the town is given a deed to a narrow strip of land connecting his property with the river bank. This will be an instance where conditions within the town may be made to fit a fine-haired theory. What, then, will be the status of the city user? He is a riparian owner in fact and a preferred user. Because he and his neighbors join together, and by combined efforts build a water system that will supply all within the limits of the city, there is no reason that we should assume that the character of the use has changed in any

way. The total demand on the stream may increase because of the growth of the town. The character of the use remains the same. Regardless of this, cities and towns have often been compelled to undergo many hardships because the Courts have generally given private enterprise the advantage wherever the two classes of claims or rights have been in conflict. As soon as a study of preferred uses is undertaken, we must begin to readjust some of our former views. If we say that the first user, as to time when use began, has the first right, should this principle apply to uses of a preferred character? When a preferred use expands and becomes a detriment to a use not preferred, what shall be done in order that the higher use may have protection and the lower use be not damaged or even confiscated? In some States, the preferred users are always protected, regardless of the effect thereof on others. This is probably a wise rule when all users understand the situation from the beginning, and when all investors know that the principle will be carried into effect. Probably the better plan is to have an understanding, with users not preferred, to the effect that as the preferred use grows, the inferior users shall relinquish certain rights and be compensated therefor at a fixed rate, regardless of the time the readjustment may take place, or the demands that may be made on the stream by the preferred user. This should all be done under public supervision. Where water is wasted during times when the supply is inadequate for all, the public should interfere. A city should be permitted to expand its use without being compelled to compensate a user who wastes water. Waste, during times of scarcity, should never occur, under any well-regulated system of public control. By waste is meant a loss of water through carelessness, or such loss as may be prevented by a reasonable expenditure of money in repairs or improvements. There are losses which cannot be prevented, and these we cannot well consider.

Where irrigation is essential to agriculture, the use of water therefore should be preferred, to a limited extent at least. Power can be developed in other ways, and power plants may be located where they will not interfere with the use of water for irrigation. The ideal relation of these uses, geographically, would require the power plants to be located up stream from the irrigation works. For instance, in a mountainous country, the greatest fall is obtained near the headwaters of the stream. This is favorable to water-power plants. If a

reservoir can be constructed on a main stream to equalize the flow of water and produce a constant head for the power plant, which then discharges into a second reservoir where the water may be stored until needed for irrigation, no interference will result. It is impossible to maintain a power plant at maximum efficiency on a reservoir where water is stored for irrigation. Irrigation demands the maximum use of water in a short period during the summer, generally speaking. The maximum demand for power often comes during the winter. The head available, at the reservoir which stores water for irrigation purposes, is at a minimum at the close of that season. The supply of water, therefore, would be irregular for the power plant, and the head would not be uniform. Some of the most disastrous litigation in foreign countries, where irrigation is a necessity, has been brought about by the establishment of power plants which retard or prevent agricultural development.

That preferred uses should be recognized and protected, all must admit. When all facts relating to uses on a single stream are brought together, the administrative officer can determine the relative rights based on other considerations; after which, giving these higher uses appropriate weight, the final adjustment can be made. It would seem difficult to balance these rights, keeping two principles in mind at the same time, but in practice this is not as troublesome as it is in theory. Preferred uses are always comparatively small, and the public generally consents whenever a public authority makes a decision which protects them.

e.—Beneficial use, and the extent thereof, at the time the determination of rights takes place, shall be the measure, the limit, and the basis of the right. Where rights and claims are in an inceptive period, the public record relating to the determination may properly mention them, and all prospective users be given a reasonable time to complete the application of water, under the plans submitted to the office of public record. These incomplete rights should be determined later, as the water is applied to beneficial purposes.

The principle of beneficial use is an important one. When it is applied, every user is restricted to a specific volume, as a maximum, and he can only use so much thereof as can be applied beneficially. Under the Court decisions of the country, generally, claimants are given specific allotments of water, and in many places they are per-

mitted to sell what they find they do not need. This practice breaks down nearly every important principle relating to stream administration. The resulting condition is almost impossible, from the standpoint of the water user. Under the principle of beneficial use, a power plant, for example, is not entitled to the uninterrupted flow of the stream, regardless of the needs of the plant. Under the doctrine of riparian rights, as it is interpreted frequently, a water-power plant can require the entire flow of a stream to reach the plant, regardless of how much thereof can be used beneficially. This permits the plant to waste as much as it pleases, and to bring damage suits, even when it is not injured by users above. If the right of the power plant is limited by the volume it can use beneficially, the owners thereof have no control of or interest in the volume that the stream furnishes in excess of the capacity of such plant.

f.—A declaration of abandonment of rights to use water must be made where water is not used for a certain specified period. The application of this principle makes it impossible for those having water rights simply to hold them without using the water. This rule is incorporated in the statutes of many Western States, yet its practical application is always hampered, because the administrative officers do not generally exercise control, and it is indeed seldom that public, rather than private, interests are upheld. We have a network of theories relating to abandonments, and though these are interesting to study within doors, they are worth but little in the field. The Courts have held almost unanimously that "to constitute abandonment there must be a concurrence of act and intent, the relinquishment of possession, and the intent not to resume it for beneficial use, so that abandonment is always voluntary, and a question of fact".* This means that a user, who has been accorded a right to apply water to some beneficial purpose, may cease all use for as long a period as he may desire, and the actual abandonment will only take place when he admits the same. This is a poor rule. When the use ceases for a limited period, this should be taken as evidence that the water right is of but little value to the former user, and he should be deprived thereof. The public is concerned in use, not in speculation. A definite period, quite brief, should be fixed by law for the interval of non-use. Where the use of

* Wiel, "Water Rights in the Western States."

water is suspended, for such a time, as a matter of convenience rather than of necessity, the right should be canceled, and those who can apply the same beneficially should be favored. The application of this principle emphasizes the principle of public control, so that it has a secondary as well as a primary value.

There are other minor principles that might be discussed. The more important general principles have been already referred to, and a law, State or National, which embodies them, will result in an equitable administration of streams. These principles are not difficult to understand. Injustice is always done when they are obscured by statutes which relate only to procedure, and Court decisions that deal with fine-haired theories. It may be said that, had these wise principles been first incorporated in the law, the Courts might have been able to "adjudicate" the various rights in a satisfactory way. The West has tested this procedure thoroughly. All such trials have resulted in failure. Reference will be made to a single Court decision, made after the doctrine of riparian rights had been abrogated. Where irrigation is practised, a large number of claimants are generally represented. Table 1 is given, without referring to any particular State or stream, yet the data are taken from the Court records. Instead of giving the names of the water users, or the ditches, the writer will simply list the adjudicated rights in order of "priority" numbers.

The adjudication was made by a judge who took pride in his ability to frame water-right decisions. If this decree was based on any principle, the latter has been carefully concealed. An examination of the table will show that there is no relation between the area irrigated and the quantity of water allotted. It would seem that the Court permitted all kinds of testimony to be given. That a large part of this was erroneous, has been demonstrated by later investigations. Surveys have shown that the areas claimed to have been irrigated are inaccurate in practically every case. In an action of this kind in the Court, the facts are often gradually concealed by a record which contains quibbles of attorneys, statements of witnesses that are valueless, rulings and motions that are to no purpose, and testimony which has no bearing on the questions to be determined. Often the record becomes so voluminous and obscure that the Court is afraid of it, and the case is indefinitely delayed.

TABLE 1.

Priority number.	Acres irrigated.	Allotment of water, in cubic feet per second.	Area, per cubic foot per second (Column 2 ÷ Column 3).
1	160	8.65	18.4
2	1 200	67.03	17.9
3	800	30.00	26.7
4	150	12.20	12.5
5	800	37.40	21.4
6	400	22.32	17.9
7	3 300	33.55	99.0
8	300	28.00	10.7
9	300	8.40	35.7
10	200	17.90	11.2
11	300	2.35	127.0
12	300	2.25	133.0
13	160	3.6	44.5
14	......	32.0	......
15	600	12.32	48.7
16	160	3.60	44.3
17	1 500	26.50	56.6
18	1 160	84.45	13.7
19	21 000	210.00	100.0
20	800	25.00	32.0
21	2 000	60.10	32.2
22	3 000	78.11	38.4
23	100	10.50	9.5
24	160	17.50	9.2
25	800	12.00	66.8
26	4 000	45.72	89.1
27	1 200	25.00	48.0
28	......	13.00	......
29	100	10.00	10.0

In another case a State made an attempt to compromise between public control of streams, through an engineering administration, and Court supervision. Under this plan, the engineering administration is to obtain the facts and the Court is to make all decisions. This compromise was not accepted by the engineers leading in the movement without protest, but it was the best that could be obtained. This law has been tried out. The facts were determined on a single stream and all its tributaries. Complete tabulations and other needed information were submitted to the Court. This basic testimony has been in the hands of the Court for some 7 years, and nothing has been done. It is not too late, fortunately, for the public to provide administrative officers to take charge of this important work. The change can take place without a revolution, except in methods. Many countries and some of our most progressive States have already taken this step. The Courts have not been offended. On the other hand, prominent jurists have assisted in the organization of such administrative systems.

Within the limits of a municipality, where public rights and public interests are always more or less apparent, we unconsciously acquiesce

in the application of many of the principles discussed in this paper. We admit that the city owns and operates the water system. This is public control of the water supply within the city limits. An individual water user cannot sell water to his neighbor. The individual cannot say that his right to use is limited only by the volume of water that may be discharged on his premises or adjoining property. Waste is not permitted where it can be controlled. The parallel might be carried farther. We find some of the principles recognized in national and international laws relating to navigation.

Laws should be framed for the purpose of carrying principles into effect. Health laws generally define principles. When an epidemic threatens a community, the laws give the health officers authority to act at once. They may interfere with the liberty and the pursuit of happiness of many people, yet we hear but little complaint. Principles are being applied, and their value is generally recognized. As a rule, experts in science, rather than in law, frame these statutes and, in a large measure, interpret them. They know what principles should govern, and the public is willing to permit these specialists to lead, provided the important, fundamental principles be upheld. In this work, the law is incidental. The principles embraced therein are of prime importance, and we seldom question any reasonable interpretation of the law. We are thankful that public health can be protected without the necessity of preliminary litigation. We are thankful that these officers can act without great danger of an injustice being served at a critical time.

Engineers, who are placed in responsible charge of streams, belong to the same category. In fact, they must co-operate with the health service and with other administrative officers. The law, to them, consists of public directions as to their duties and, above all, plain definitions of the governing principles.

The administration of streams lies essentially within the field of the engineer. The engineer understands and appreciates the physical problems that are encountered along every watercourse. He must measure the discharge of the streams and the capacity of the diversion works. He must determine the value of water-power plants and ascertain the area and value of irrigation lands. He must estimate the volumes of water that municipalities require and appreciate the relation that should exist among these various uses.

Having all field data before him, and understanding the history of governments which have solved the problems presented, he should be able to frame laws defining the principles which should apply. These laws should be brief, and the principles should be defined in such terms as to leave no doubt as to the meaning of the phrases used. The principles should be stated first, because there is no excuse for an administration until these have been selected and defined. The officers provided in the statute then have a purpose. The foundation for an organization and for just compensation has been laid. No relief has come in any country until those understanding the physical problems have led in a campaign for laws embracing essential principles. The history of the Old World is being repeated in the New. The engineer receives but little financial reward for service of this kind. It is possible that he may secure more remunerative employment under a system which gives no protection to legitimate water users. The Profession, as a body, has never avoided a plain duty because of mercenary motives. Our great engineers are loved and remembered for the service they have performed, not because of the compensation they have received.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE CONSTRUCTION OF THE KLONDIKE PIPE LINE.

By W. W. Edwards, Assoc. M. Am. Soc. C. E.

To be Presented September 2d, 1914.

Synopsis.

Recent discussion of Alaska and the determination of the Government to undertake development work there has prompted the writer to submit the following paper, as describing conditions under which one job of heavy construction in the Far North was carried out. Detailed costs, he regrets to say, were not available, but wages and conditions are described and, whenever possible, day's progress figures are given.

A detailed description of methods used in construction is given, which may serve as a guide or as a comparison for other engineers engaged in laying heavy steel pipe.

The paper describes:

(1) Conditions of climate and geography peculiar to the Far North and affecting engineering work there. In particular, the perpetually frozen ground is mentioned.

(2) Details of construction.

(3) The difficulty of getting satisfactory foundations.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

(4) Points disclosed in the test and operations of the pipe line.

The Klondike pipe line was built during 1907 and 1908 by the Yukon Gold Company, at Dawson, Yukon Territory. It is an inverted siphon crossing the Klondike River Valley, and was designed for a capacity of 125 sec-ft., which figure, however, has been slightly exceeded in operation. By reference to the profile, Fig. 1, it can be seen that the maximum head on the pipe line is about 1 100 ft. The total length of 15 760 ft. is made up of: 2 390 ft. of wood stave pipe, 49 in. inside diameter; 5 850 ft. of riveted steel pipe, 49 in. inside diameter; and 7 520 ft. of lap-welded steel pipe, 43 in. inside diameter. The steel pipe ranged in thickness from $\frac{3}{16}$ to $\frac{11}{16}$ in., and came in lengths of from 15 to 30 ft. It is the largest of several inverted siphons along the line of the Main Ditch System of the Yukon Gold Company, 70 miles in length, which supplies water for hydraulic mining operations on Bonanza Creek, near Dawson.

The location of the pipe line was determined chiefly by two conditions: It was neces-

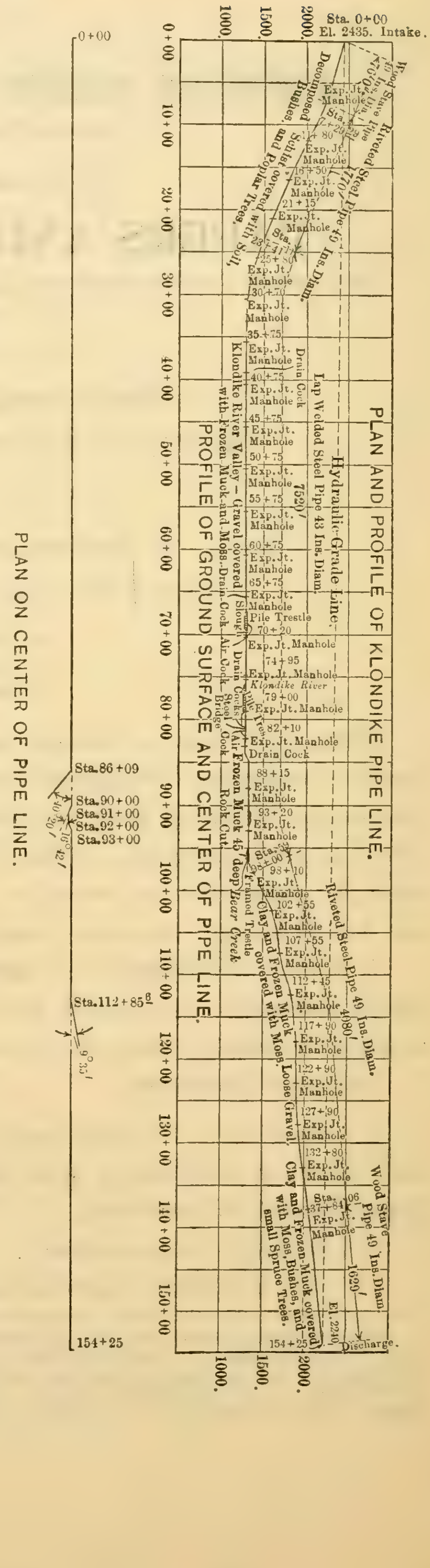

FIG. 1.

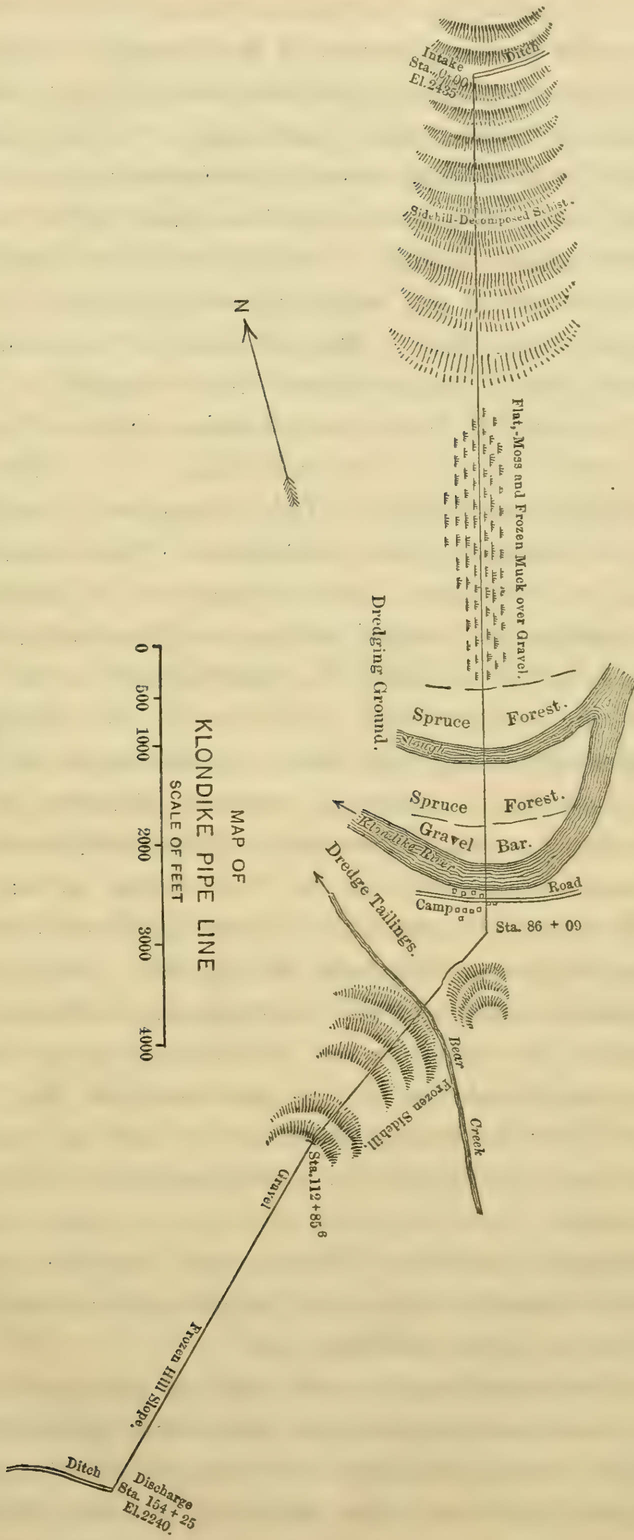
MAP OF
KLONDIKE PIPE LINE
SCALE OF FEET
0 500 1000 2000 3000 4000
N
Intake
Sta. 0+00
El.2455
Ditch
Sidehill-Decomposed Schist.
Flat,-Moss and Frozen Muck over Gravel.
Dredging Ground.
Spruce Forest.
Slough
Spruce Forest.
Gravel Bar.
Klondike River
Road
Camp
Sta. 86 + 09
Dredge Tailings.
Bear Creek
Frozen Sidehill
Sta. 112+85.6
Gravel
Frozen Hill Slope.
Ditch
Discharge
Sta. 154 + 25
El.2240.

FIG. 2.

sary to cross the valley below the junction of Bear Creek, and also to keep above the dredging ground of the Canadian Klondike Mining Company. These two determining features left little room for adjusting the line to fit the ground conditions, but it is probable that the location adopted was as good as could be obtained in any case.

As may be noted from the profile, the intake end of the pipe is on a comparatively steep side-hill. This hill is of decomposed schist covered by a foot or two of soil which supports a growth of bushes, poplars, and a few spruce trees. The flat valley of the Klondike is here about 5 000 ft. wide. It is covered with a blanket of Arctic moss from 1 to 2 ft. in depth. Below this there is a layer of frozen "muck", or vegetable mould, which is generally about 5 ft. thick, and below this is generally found gravel. This formation is peculiar to the North, and will warrant a little explanation. The frozen "muck", or "frost", as it is sometimes called, is made up of decomposed roots, moss, tree branches, etc., mixed with some little rock and sand, and the whole mass is frozen solid. The moss blanket on the surface protects it from the summer heat, and whenever this blanket is undisturbed the ground beneath the moss remains frozen the year round. If the moss covering is removed, however, allowing air and water to act on the frozen muck, it soon melts. This condition may be found in almost every river valley in the Far North, as well as on the sheltered hill slopes. In places the "frost" has been built up by the seasonal thaw not penetrating fully through the winter's freezing, and extends to a known depth of 200 ft. and unknown further depths. This condition makes the question of foundations quite serious. The "frost" is a good foundation if it is not disturbed, but, if disturbed, the structure placed in it must be frozen in and the surface mossed over again to keep out the air and water absolutely. If this cannot be done, the structure must be carried through the frozen muck to good gravel or bed-rock foundation. Frozen muck was encountered all the way across the Klondike Valley and in spots along the pipe line up the side-hill as far as the discharge box.

Only two horizontal angles were made in the pipe line, with the exception of those used in making a short offset around the base of a hill. Manholes, drains, and air-cocks were provided wherever necessary. Cast-steel expansion joints were put in every 500 ft. to allow for the expansion and contraction due to the extreme variation in tem-

perature, from 80° Fahr. in summer to — 70° Fahr. in winter. One three-span steel bridge, 295 ft. long, was necessary in crossing the Klondike River, and a total of several hundred feet of pile trestle or framed trestle were put in at various places along the line. The aim was to have the pipe supported at least once in every 16 ft. Whenever the conditions warranted it, as for instance, when good foundation was deep, trestle bents were built every 32 ft. and king-post trusses were put in to give the intermediate bearing. In good material, sills were merely laid in the bottom of the pipe trench and on these the pipe was supported. Whenever the foundation was within 3 or 4 ft. of the pipe, crib supports were used; if greater than 4 ft., framed bents were used.

One anchor was put in at least every 500 ft., about midway between expansion joints, and, where necessary, they were used more frequently. Each anchor consisted of a cable wrapped around the pipe in the form of a clove hitch, the ends being made fast to two "deadmen" on opposite sides of the pipe. Each "deadman" was set in the ground about 6 ft. below the surface.

On the side-hills drains were dug at frequent intervals to lead off seepage and leakage. At every drain a bulkhead was put in which fitted closely to the pipe and effectually intercepted the water.

In many ways conditions were unique for heavy construction. Common labor was paid $4.00 per day and board, which meant a total of about $5.50 per day. The working season was short, being limited to 5 months—May 1st to October 1st. Some kinds of equipment were plentiful in the country, notably steam engines and boilers, but in other lines it was sadly lacking. A well selected stock of equipment was ordered for the job, but breakages and unforeseen difficulties sometimes caused awkward situations, in view of the distance from a base of supplies.

Power was furnished from the Yukon Gold Company's hydro-electric plant on Little Twelve Mile River, about 50 miles from the pipe line. It was delivered at the main transformer-house at 33 000 volts, and there stepped down to 2 300 volts. From there it was distributed at this voltage to the transformer banks at the several compressors and at the machine shop, where it was stepped down to 220 volts.

Construction was begun in 1907. The right of way was cleared and a small quantity of pipe trench was dug. The three-span steel

bridge over the Klondike was erected. Foundation cribs were put in the river bed, the work being done mostly while the river was frozen over and the water low. Piles were driven in the river bottom and sawed off at the gravel surface, and on them the cribs were built. These cribs were then extended to a height of about 5 ft. above high-water level and filled with rock. They were sheathed with sheet iron on the up-stream end to protect them from ice. Derricks were used in assembling the steel spans. During this season, also, a pile approach to the bridge, 400 ft. long, was built, and several other smaller pile trestles and framed trestles. Construction was halted in the midst of the working season in 1907.

In the spring of 1908 construction was begun again. It was desired to have the job completed by the following October 1st, the end of the working season. All the pipe was not then on the ground, and what was on hand was not continuous. That is, sections of pipe were missing here and there. As there were so many different kinds of pipe in the line, and as extra pieces were few, it was generally impossible to use sections of pipe interchangeably, therefore work could be started only at certain places. In the course of the work, it was necessary to start laying pipe at four different stations. An error in surveying would lead to awkward consequences where any of these headings joined, as well as at angle points in the pipe line. As there was no heavy machinery or plate for making pipe of this size in the country, and there would not be time to send "outside" for extras, it was necessary to be very careful in surveying.

All the pipe on hand was first measured up carefully and a relation established between actual measurements and those given on the blue prints. An offset line was then run paralleling the located line at a distance of 20 ft. As the seasonal thaw had loosened the original hubs so much that they could not be relied on, measurements were begun from a point on one of the bridge piers. Hubs were driven about 99 ft. apart, and measurements were taken from plumb-bob strings held from tripods. This was done on the side-hills as well as the flats. A standard pull of 15 lb. was used, the same as had been used in measuring up the pipe on hand. Corrections were made for temperature variations. Levels were run over the hubs, and horizontal distances were computed from the slope measurements and elevation differences. This system resulted in giving a first-class base line, which could be

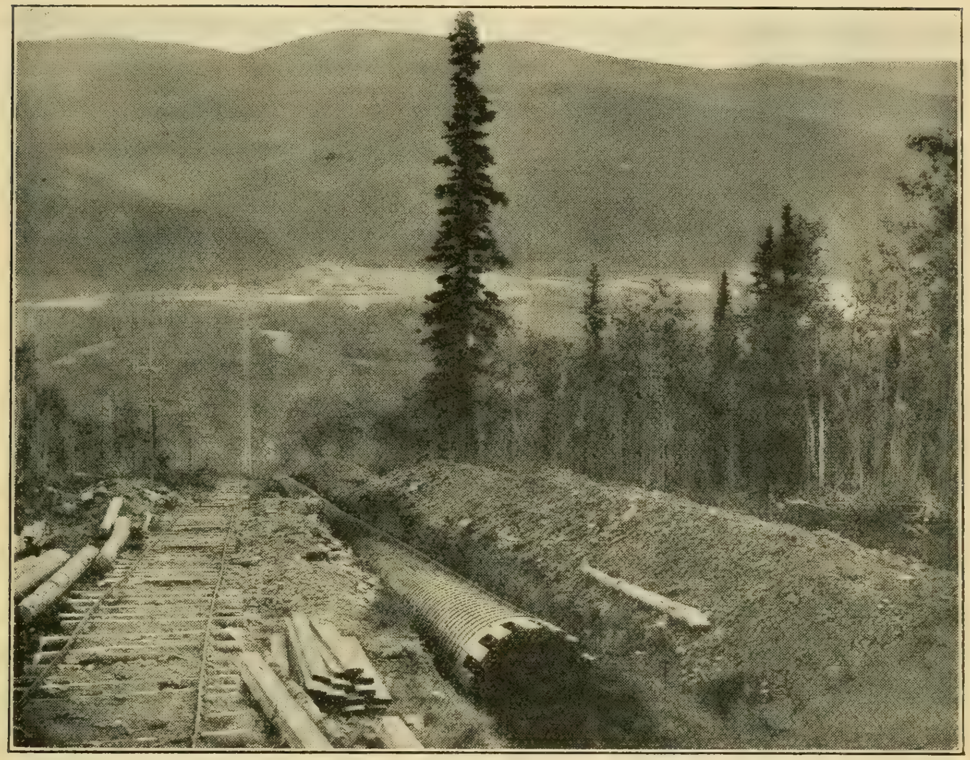

Fig. 3.—View of Intake Side-hill, Looking Toward Discharge.

Fig. 4.—Klondike River and Bridge, Looking Toward Intake.

absolutely relied on in setting grades or starting points. Grades were "shot in" by transit. Points were set along the edge of the pipe trench, and were transferred to the bottom, when needed, by using a large wooden square and a carpenter's level. Bench-marks were set every 50 ft. in elevation. Levels were determined by running ahead for half a day and then tying back to the starting point.

In construction, it was frequently found advantageous to depart from the profile in order to avoid excessive excavation, and in these cases grade changes were made by shifting angle points.

Pipe was distributed along the line by a tramway. The cars were pulled by horses across the flat and by hoists on the hills. Of these hoists, two were run by steam, and one by electricity. Signals were given by use of a wire attached to a spring pole. A telephone system also helped in signaling and general communication. Turn-tables were installed on which to change the sections of pipe end for end when necessary. Steel-frame cars were used. In hoisting, a bridle was attached to the end of the pipe, and the cable was shackled to this bridle. When the pipe was at the spot desired, the cars were chained to the rail, and the pipe, with the hoisting cable still attached, was rolled into the ditch. This held the pipe from getting away on the side-hill, and the cable always had so much spring that no harm was done to the engine.

The fitting-up gang used ratchet jacks in fitting up the pipe. The section was first slid down to the last section already connected up and then turned until it was right side up. The "front", or free end, was then raised until the top holes at the back end could be caught by drift-pins to the last section. Guide-straps were then bolted on at the joint, and the front end was lowered gradually. As the front end came down, the joint was sledged and the guide-straps were tightened as necessary. When the joint was entered all around, a ratchet turnbuckle was attached and used to pull it together. Fitting-up bolts were put in every third or fourth rivet hole. Four or five sections of pipe per 10-hour shift was an average for the fitting-up gang.

Riveting, caulking, and drilling were done with pneumatic tools. Air was furnished by 8 by 8-in. Clayton air compressors run by 15-h. p., 3-phase motors. One air compressor would furnish air enough to operate a riveting hammer, caulking hammer, drills, and air dolly, with not much to spare. The compressor outfits were mounted on

platforms and moved from place to place as desired. The riveting of the holes in the bottom of the pipe was done from the inside, and of the upper holes from the outside. Air dollies were used in holding on inside the pipe and spring dollies on the outside. Each riveting gang consisted of a riveter, bucker-up, heater, and passer. An average day's work was from 160 to 180 1-in. rivets, when conditions were good. Fitting-up bolts were used liberally, and the joint was well sledged before driving each rivet. All castings were hand-riveted to the pipe, the riveting being done on the inside.

The expansion joint castings were frequently found to be too large for the pipe. In such cases the pipe was heated by a blast lamp and belled out with hammers to fit the casting. Rivet holes in all castings were drilled in the field after assembling. All caulking was done with rounded-edge fullers, the square tool being dispensed with. The riveted pipe was caulked on the outside only, the lap-welded pipe was caulked both inside and outside. Rivets were inspected with a 1-lb. tapping hammer.

The pipe was coated on the outside with a red mineral linseed oil paint and inside with a quick-drying black paraffin paint.

The wood stave pipe was of California redwood staves thoroughly seasoned. It was laid in a trench 2 ft. deep, and, when completed, was back-filled to a depth of half the diameter of the pipe, so as to give a good bearing.

The intake penstock was equipped with overflow and turn-out, as was also the discharge box. The discharge box was located on a shaded side-hill on frozen clay. This condition made necessary considerable cribbing to protect it from the sloughing of the clay when it thawed consequent on the removal of its moss covering.

Timber was secured from adjoining leases and also from river drives. It was necessary to do considerable work to protect the river banks from undercutting in the vicinity of the bridge, as they were of loose gravel, loam, and moss. Piles were driven and shear cribs put in along the river bank above the bridge. Protection cribs were also put in above the pile bents on the bridge to take the brunt of the ice pressure and to break up the ice stream.

Frozen muck, reaching to a depth of 45 ft., was encountered between Stations 84 and 90. Shafts were dug to gravel foundation and trestle bents were put in at intervals of 32 ft. Between Stations 90 and 93

a heavy cut through dredge tailings and solid rock was necessary. This was just at the base of a hill which was covered with frozen muck. The digging of this section was delayed until as late as possible so that the extreme cold would keep the overburden on the slope above from thawing on exposure, and consequent sliding.

A small spot of frozen clay was encountered at about Station 107, and the depth of good foundation being unknown, a corduroy of spruce trees was laid on the frozen clay in the bottom of the pipe trench and this was covered with about 2 ft. of gravel. A bulkhead was built around the pipe above this place to shut off all seepage and leakage, and a drain was run off to the side. The pipe was back-filled and the surface covered with moss again. Apparently, the foundation remained frozen, as no settlement was noticed.

Although work was begun at the camp about April 1st, it was not until about May 15th that all the snow was off the ground and it was possible to work to the best advantage. A night shift was put on about May 15th and continued until October 1st, by which date the pipe was all laid. The long days of the extreme northern latitude made lights unnecessary on the work until August 1st. After that, a system of arc and incandescent lights was installed, which worked very well. The missing sections of pipe began to arrive about July 1st, and from that time onward operations were rushed. A force of 350 men was used to push the work to completion. The different headings of pipe were connected up at the expansion joints. This was effected by sliding the outside ring of the expansion joint back on the pipe and then pulling it up to position after the pipe had been lined up.

A tabulation was made showing the space to be left open at the expansion joints at various temperatures, so that they would not close during the hottest or open in the coldest weather. This was followed in setting the joints. Gauges were wired on at each expansion joint so that unequal expansion or contraction could be noted and steps taken to prevent it. The coupling bolts for each expansion joint were set so that the lap in each joint could not be less than 1 in. Reference points were also put in on each 500-ft. section of pipe, so that movement could be detected. Anchors were put in at every angle point. These were generally either bulkheads of timber against the pipe, used as thrust anchors, or cables around the pipe attached to "deadmen." When possible, at the angle points, a timber crib was put completely

over the pipe and filled with rock. The high cost of transportation and the character of the foundation made concrete anchors out of the question. The only timber available was white spruce, which was soft and brittle, and had to be used with a high factor of safety in all structures.

Some trouble was experienced from the lap-welded pipe. Two makes were used on the job. One, an American make, was perfectly satisfactory, but the other, a German make, was not. One section of the second kind cracked along the weld for a distance of 10 ft. on being dropped into the pipe trench. This was patched and reinforced, and gave no trouble in operation. Another section of this pipe burst in testing under 835 ft. head. Examination showed a flaw in the metal. When it burst, the water was shot out to the side of the pipe line and little damage was done. This was the extent of the trouble with the line in testing.

In operation, the quantity of water flowing through the pipe fluctuated considerably. As the difference in elevation between intake and discharge was 195 ft., this variation caused great fluctuations in the water level in the pipe at the intake end, there being no regulating gate at the discharge end. When little water was flowing, it ran in a very rapid stream down the bottom of the wood stave pipe at the intake end. This had the result of rapidly wearing out the bottom staves, so that considerable patching had to be done.

Bulkheads were put in at each end of the pipe and all other openings were closed as soon as the pipe was thoroughly drained. Thus the air circulation inside the pipe was shut off and the subsequent snow formed a blanket which protected it from the extremes of the winter temperature. It was found that with an air temperature of — 30 or — 40° Fahr. the temperature inside the pipe was only about — 5° Fahr. In operation, the pipe is drained at the close of the working season, about October 1st, and opened again about May 1st.

The peculiar and therefore the most interesting feature of this work was, as noted before, the study of foundations and the best styles of substructures to be used in the frozen ground. In places in the North, piles of saw-dust have been placed on the moss, and buildings put on this foundation. When water is kept away, this makes a very good foundation, as the saw-dust protects the moss and the frozen ground below from heat and consequent thawing, thus insuring

stability. On this same principle, in running a road or railroad grade across frozen ground, it is best not to cut but rather to make fills on top of the moss. Thus, on a railroad grade near Nome, a fill was made of peat cut from the surface of the tundra. When dried out and packed this gave no more trouble—until it burned up—whereas a cut in frozen ground is but an invitation to perpetual trouble from slides and settlement. In flume substructures, on the Yukon Gold Company's line, on frozen side-hills, cripple bents were tried and discarded in favor of level mudsills set in the "frost", about 2 or 3 ft. below the surface at the down-hill end. These were mossed over and remained frozen unless excessive leakage thawed the ground.

Mr. A. C. Strong, of the Yukon Gold Company, developed a type of structure which suited the foundation very well. He used piles driven into holes previously thawed out by steam points. The steam point is a strong pipe, at one end of which steam is introduced. The opposite end is rounded and a small hole left through which the steam may escape. This point is started into the frozen ground and is pounded down with a hammer as fast as it thaws through the "frost" in its way. These steam points were developed and used to considerable advantage in mining the frozen gravel. In substructure work, the steam point is used until the "frost" is thawed to the desired depth. It is then removed and a pile driven into the thawed-out hole. As the surface moss is disturbed hardly at all in this process, the pile immediately freezes in and remains in this condition. At last reports, pile substructures, such as these, were giving good service. This appears to be the cheapest form of substructure as yet devised for use in ground frozen to any considerable depth.

The writer has seen but little in technical literature dealing with construction in the Far North, and hopes that some discussion may be stimulated by this paper, especially on the subject of structures founded on frozen ground.

The foregoing work was done by day labor by the Yukon Gold Company, Mr. O. B. Perry, General Manager; C. A. Thomas, M. Am. Soc. C. E., Resident Manager; H. H. Hall, Assoc. M. Am. Soc. C. E., Superintendent of Ditch Construction, and in general charge of surveys; Mr. C. G. Newton, Superintendent of Pipe Lines; and the writer, Division Engineer and General Foreman.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE CONSTANT-ANGLE ARCH DAM.

By Lars R. Jorgensen, Assoc. M. Am. Soc. C. E.

To be Presented September 16th, 1914.

Synopsis.

There are two features which distinguish this type of dam from the ordinary arch dam. These are (1) "Economy of Material", and (2) "Ability of the Dam to act as an Arch, even Close to the Foundation, to a Much Greater Extent than the Ordinary Arch Dam." It is understood that the canyon is wider at the top than at the bottom, which is generally the case.

The first claim, "Economy of Material", is proved in the following manner. It is shown that any arch slice closing the gap between two abutments must subtend an angle of (133° theoretically) about 120° for greatest economy of material.

The constant-angle arch is designed to comply with this law as closely as possible at any elevation from crest to foundation. The length of the up-stream radius is decreased toward the foundation in the same proportion as the canyon becomes narrower, thereby keeping the angle subtended by the arch at the most economical value. In the case of an ordinary arch dam built in a V-shaped canyon, the angle subtended by the arch is rapidly getting smaller toward the lower elevations, and therefore the building material is placed in a less

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economical position and the arch soon becomes so flat that it cannot act as an arch, but the load is carried by shear and cantilever action almost exclusively.

To prove the second claim, attention is called to the fact that the deflection of an arch when loaded is proportional to the square of the length of the up-stream radius. This radius may be several times shorter at the foundation than at the crest of the dam in the constant-angle arch design. If the radius should happen to be, say, four times shorter at the foundation than at the crest, the deflection required at the foundation would be sixteen times less than at the crest, using the same unit stresses. In such a case, the arch would be able to support sixteen times as much load at the bottom as an ordinary arch having a constant up-stream radius.

In order to obtain equal safety at all points, it is necessary that the thickness of the arch should be made the greatest in the middle, and diminish toward the abutments, especially toward the lower elevations. This is on account of the fact that an arch dam when loaded acts like a long column in compression, and such a column is weakest in the middle; besides, the central portion of the arch moves back and forth according to different load conditions, and for that reason alone the unit stresses used for this portion should be kept lower than for the more stationary portions of the dam.

It is also pointed out in the paper that the initial stresses (Poisson's ratio), due to the water pressure and to the weight of building material, can be utilized to carry a portion of the load, and that these stresses will carry the greatest proportion of the load in arch dams designed according to the constant-angle arch principle.

An approximate short-cut method for finding the proportion of the load taken by the arch and by the cantilever, respectively, is given, also some information as to the maximum foundation pressure at the toe, and the shear existing at the foundation. Finally, a few practical examples are described.

It has been the object of the writer to bring out a dam design especially adapted to high and comparatively narrow canyons. For such a condition, the constant-angle arch dam will generally show a saving of material of 33% or more over an ordinary gravity dam, and at the same time it will possess a factor of safety more than twice as great as that of the gravity dam.

This method of arch dam design shows how it is possible to make the dam take most of the load, acting as an arch, leaving only a small portion for cantilever or gravity action.

There is no single type of dam which can be readily adapted to every site, therefore a number of types have naturally been developed and applied to satisfy the different demands as these have presented themselves. This statement applies to the type of arch dam to be described in this paper, as well as to any other.

All masonry dams must be built on good rock foundation, with the exception perhaps of those of the Ambursen type, the requirements for which are not so exacting. Where the dam site is narrow and more or less V-shaped, a single arch dam of the type to be described can be built to advantage. How narrow the site must be cannot be stated in general terms, for a few calculations will have to be made to show whether there is economy in using the arch dam rather than any other type. Many dams having gravity sections have been built curved in plan, this curved form giving them considerable extra strength at the crest, especially when the arch subtends a large angle. However, the ordinary arch form gives but little extra strength at the bottom of a V-shaped canyon, because, no matter how large an angle is subtended by the arch at the crest, this angle has become small at the bottom. In other words, the arch cannot carry much load on account of its flatness. This non-action of the arch at the bottom the following method of design seeks to eliminate, besides giving a most economical distribution of material. It does not matter whether a gravity section or a much thinner one is used, the arch can be forced by the design to take up the greater portion of the load. That this is the most economical way of loading the structure should be readily admitted, as arch stress is distributed nearly evenly over the whole section, whereas the stress due to gravity action is very poorly distributed, being a maximum along the down-stream face, and diminishing to approximately zero at the up-stream face. These two stresses are in two different planes, 90° apart, thereby tending to give lateral support to each other, as both act at the same time.* This indicates

*This conclusion can be fairly well proved by the fact that when testing a cube or column of any material in compression, it swells laterally in a plane 90° from the plane of the compressive force. With lead, this is quite apparent. If this swelling be prevented, the crushing strength of the material will be materially increased.

that it is not economy to eliminate entirely the cantilever or gravity action in an arch dam, even if this were possible, as it acts like the hooping in a column, giving lateral support to the dam body. The arch action, however, must predominate, on account of the more economical distribution of stress over the section, and also in order to eliminate the large shear action otherwise introduced by the force causing the bending of the cantilever.

When an arch dam is loaded, the length of the arc is bound to become shorter, due to the deformation of the material caused by the axial stress. The only way the structure can compensate for this shortening is by deflecting down stream. If the canyon has a fairly regular shape, this deflection will be greatest at the crown of the arch and will decrease to zero at or near the abutments. This deflection of the crown introduces cantilever stresses which ordinarily reach their maximum values in the middle section, where the dam is highest and the deflection is greatest. How the load divides itself between the arch and the cantilever depends on their relative ability to carry load, and it is easily seen that if we wish to design an arch dam where the arch is to carry the greater portion of the load, we must keep the arch deflection down to a minimum, and especially at and toward the bottom, where the dam is fixed in position and cannot move. Then, too, the arch should be designed so that the amount of this deflection decreases at a uniform rate from a maximum near the crest to a minimum, as close to zero as possible, at the foundation, in which case any imaginary horizontal arch slice deflects as if it were actually independent and free to move relatively to any other slice above or below, thereby eliminating undesirable internal stresses.

In order to obtain a preliminary dam section for any given dam site the simple formula,

$$t = \frac{P\,R_u}{q} \qquad (1)$$

can be used for finding the thickness of a sufficient number of arch slices at different elevations. In this formula, t equals the thickness of the dam, in feet, at any given elevation; P equals the water pressure, in pounds per square foot; R_u equals the length of the up-stream radius, in feet; and q equals the average stress, in pounds per square foot, of the area of the dam section (Fig. 1).

It will now be shown how to obtain the most economical arch section, that is, a section where the material is placed so as to take up the load with minimum resulting stress. From the formula, $t = \frac{P\,R_u}{q}$, it is seen that the thickness, and therefore the area, of the dam section, varies in direct proportion with the radius. The volume of concrete in any arch dam is equal to the area of the section multiplied by the length of the mean arc. The length of the mean arc can be expressed as the length of the mean radius, R_m, multiplied by the subtended angle, in terms of π,

$$\text{or, } V = \text{Area} \times R_m \times 2\,\theta \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where $2\,\theta$ is the subtended angle.

The mean radius, R_m, equals half the width, W, of the span divided by the sine of half the subtended angle (Fig. 1). Thus

$$R_m = \frac{\frac{1}{2}\,W}{\sin.\,\theta} \ldots (3)$$

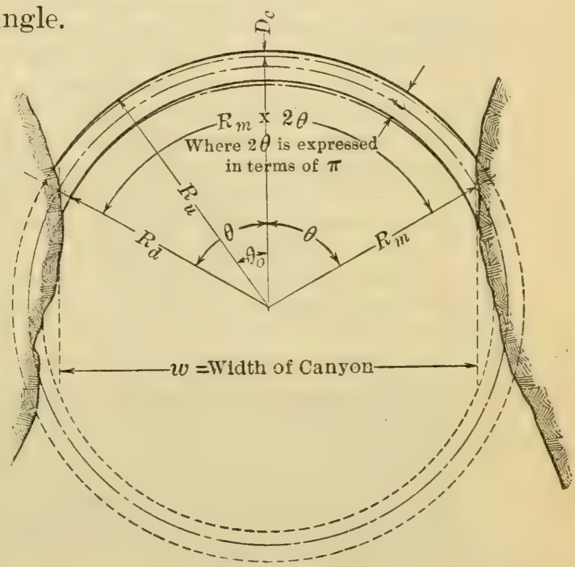


FIG. 1.

As the area of the section is proportional to the length of the radius (both to R_u and R_m), Formula 2, for the volume of masonry can be expressed thus:

$$V = C \times \frac{\left(\frac{1}{2}\,W\right)^2 \times 2\,\theta}{\sin.^2\,\theta} = \frac{K \times \theta}{\sin.^2\,\theta} \ldots\ldots\ldots\ldots (4)$$

where C and K are constants, the latter depending on the width of the canyon.

According to Formula 4, the volume varies with the term, $\frac{\theta}{\sin.^2\,\theta}$. The differential coefficient of this term, equated to zero, gives the minimum for a central angle of 133° 34′, which means that any horizontal slice of the dam has the least volume when $2\,\theta = 133°\ 34'$. In other words, the dam contains a minimum quantity of material when the central angle is kept at 133° 34′ at all elevations.

The curve on Fig. 2 shows this graphically; the abscissas represent the central angle, $2\ \theta$, and the ordinates represent the term, $\frac{\theta}{\sin.^2 \theta}$, the latter being proportional to the volume of masonry. In addition to showing the point of maximum economy, this curve also shows that, as long as the subtended angle, $2\ \theta$, is kept between the limits, 150° and 110°, the variation in the volume of masonry is very small, but that, outside these limits, the volume increases rapidly. Fig. 3 illustrates how the dimensions of successive arch-shaped slices of a dam are determined and superposed, one upon the other, to form the structure. Six elevations have been established, forming Contours

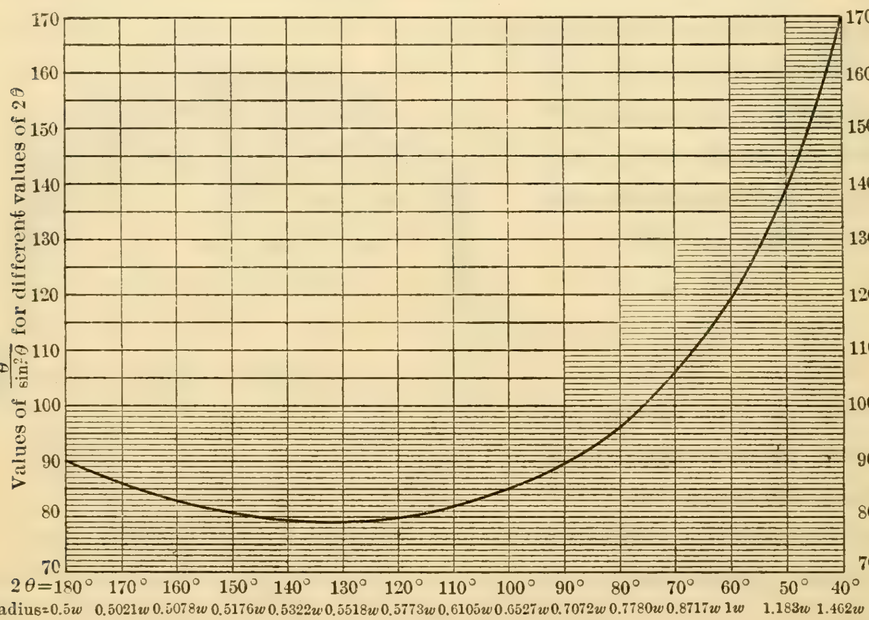


FIG. 2.

I, II, III, IV, V, and VI, Contour VI corresponding to the level of the water retained by the dam, and Contour I corresponding to the lowest water level. The distance across the canyon at the top of the dam, W_6, is measured on a map to scale. Then the mean radius, R_{m6}, which will give the dam the greatest strength with the least volume of material, is found by substituting the most economical central angle, $2\ \theta$, in the formula $R_{m6} = \frac{\frac{1}{2} W_6}{\sin.\ \theta}$. The thickness, t_6 may now be determined algebraically from Equation 1. However, for the first calculation, the length of the up-stream radius will have

to be estimated from the known length of the mean radius, but this is not difficult.

At Contour V the distance, W_5, across the canyon is measured, and the most economical radius and thickness of section at this elevation is found in the same manner as for Contour VI. The center of curvature of the arch slice at Elevation V does not necessarily lie on the same center line as the center of curvature of the arch slice at Elevation VI, as perfectly symmetrical slopes of the canyon sides will rarely be found. It is desirable, however, for practical construction and appearance sake, to try and keep as many centers

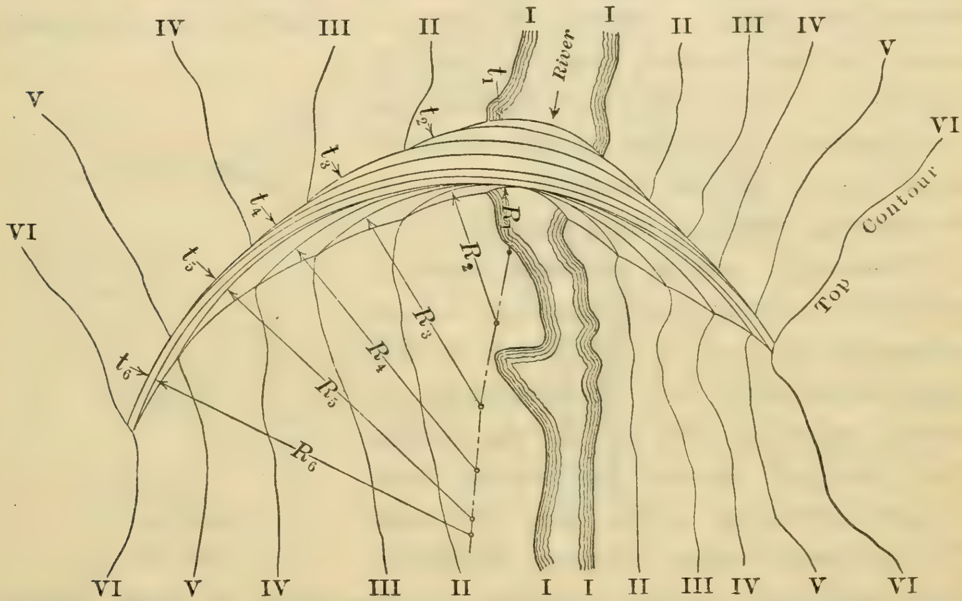


Fig. 3.

as possible on common center lines, as shown on Fig. 3. The radii and thickness at lower elevations are determined in a similar manner. It is thus seen that the main requirement which makes for high economy of material is to keep the central angle of the arch as near to the most economical value at each elevation as the contour of the site will permit. This necessitates the abandonment of the constant up-stream radius generally used in arch dams as at present constructed, and the substitution of radii of varying lengths corresponding to the widths of the canyon at different elevations. In some localities it may be desirable to face the dam with stone, and it may then be of advantage to step the faces. It will then consist of a number of

cylindrical rings superposed one upon the other. The up- and down-stream radii in such cases are ascertained in exactly the same manner as given above.

To prevent upper portions of the dam from overhanging lower portions, it will be necessary to have the thickness of the section increase from the crest toward the foundation. The proportional increase in water pressure, therefore, must be greater than the proportional decrease in length of the up-stream radius toward the foundation. The ratio of increase in water pressure is always fixed, and the ratio of decrease in the length of the up-stream radius depends on the slope of the canyon sides. If these slopes are such that at any intermediate elevation the ratio of decrease in length of the up-stream radius has been greater than the ratio of increase in water pressure, a decrease in the thickness of the dam at this elevation would result, and the structure would be overhanging, which is impracticable.

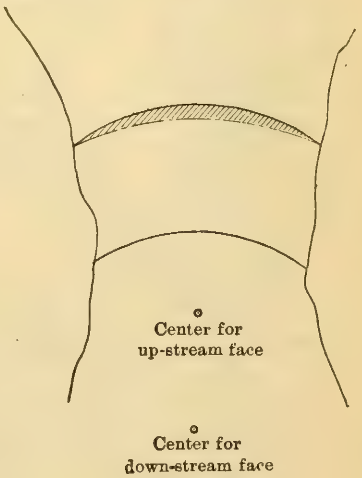


FIG. 4.

Whenever a certain thickness must be provided to prevent overhanging, it is most economical to increase the length of the mean radius above that corresponding to a central angle of 133° 24′, for the reason that a flat arch requires less material than a more curved one of the same thickness. The up-stream radius, however, should be kept small, and not have a center common with the mean radius, and of such length as to allow the deflection line (*A*, Fig. 9) to become one continuous straight line from crest to foundation. The material added is small (as shown by the cross-hatched spherical triangle in Fig. 4), as such additions are generally required only near the foundations, where the arch is short.

In the foregoing, the thickness of the different arch slices has been determined as if all the load were taken by the arch, and the dam section had no gravity action at all. How safe a dam would result depends primarily on the unit compression allowed when using Formula 1 for finding the thickness at different elevations. However, in order to obtain a dam of uniform safety for its whole height, the foregoing design must be somewhat modified to take into considera-

tion the gravity and curved beam action. The foregoing theoretical design would prove to be weakest in the middle in most cases, for the same reason that a long column held at both ends is weakest in the middle, and on account of having the highest cantilever stresses here. Whenever t is small compared with R_u, the arch when loaded is practically a long column in compression, and the length of the arch, therefore, should not be more than twenty-five times its thickness, if the material is to be highly stressed. It is true that this

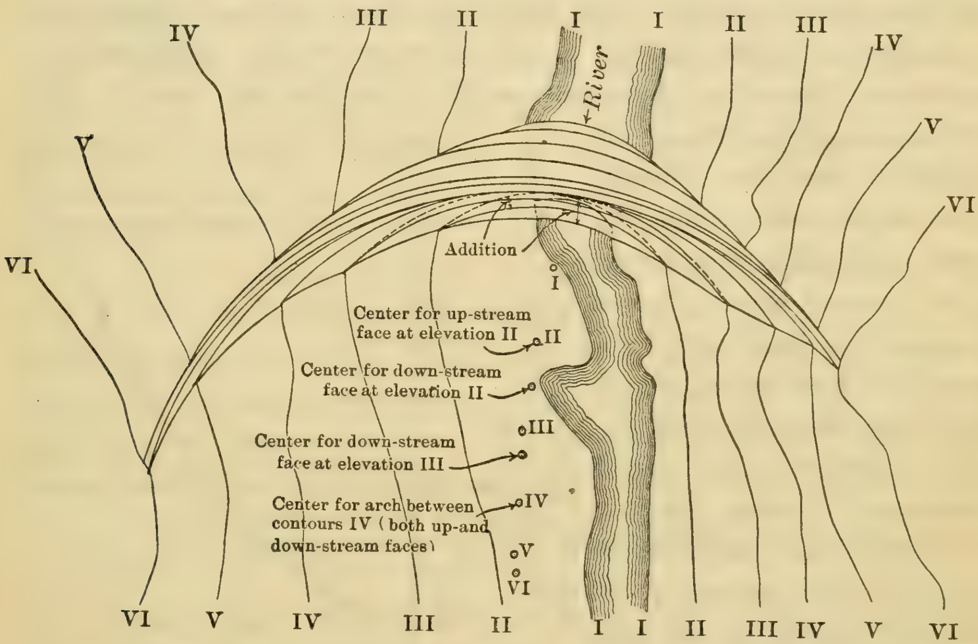


FIG. 5.

circular column is supported to some extent along one side, but this added stiffness may be largely offset by the fact that the water may not soak through the up-stream face uniformly; that is, the effect of the water pressure, in all probability, would be unsymmetrical about the center line of the dam. On a high and comparatively thin arch dam section, the resulting compression due to cantilever action and weight of material above may become excessive near the foundation, requiring some additional material along the down-stream face toward the foundation, as shown by Figs. 5 and 7. On account of the necessity for this additional material, the use of a smaller central angle than the theoretical 133° might actually be more economical, and 120° or even less, in a case like this, might give very satisfactory results (generally greater at the crest and smaller toward the foundation, depending

on local conditions). By decreasing the central angle from 133° to 120°, the volume of material is increased 1% (Fig. 2), but, at the same time, the thickness has been increased 6 per cent. For angles much smaller than 120°, the volume of additional material required increases at a greater rate than the thickness, and therefore a lower limit for the economical value of the central angle is soon reached, and extra material will then have to be added. The thickness of this added material should decrease vertically from a maximum at the foundation to zero at some higher elevation, and horizontally from a maximum in the middle, or the point where the deflection is a maximum, toward the abutments, as shown by Figs. 4, 5, and 7. This thickening of the dam in the middle to take care of cantilever stresses also stiffens the arch materially, considering it as a curved beam. It acts as such to a large extent toward the foundation, where t is large compared with R_u, and therefore this added thickness is doubly justified.

$$CC_c = \frac{\frac{2 \sin.\theta}{\theta}\left(1-\cos.\theta\right) + \frac{1}{2}\left(\cos.2\theta - 1\right) + \left(1-\cos.\theta\right)}{\frac{3\theta}{\sin.\theta} + \cos.\theta - 4}$$

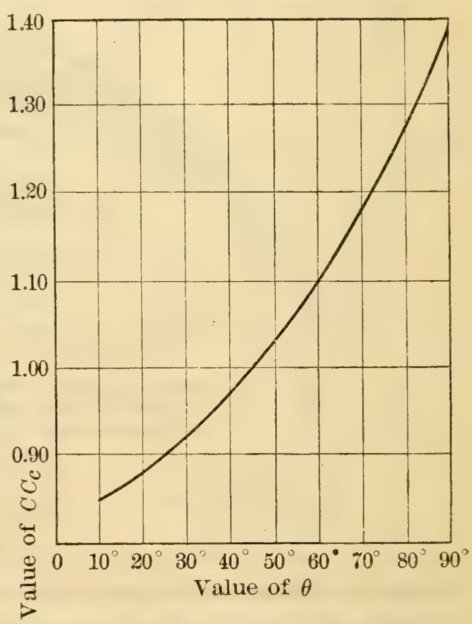


FIG. 6.

Approximate methods for finding what proportion of the load is taken by the arch and by the cantilever have been thoroughly discussed in the paper entitled "Lake Cheesman Dam and Reservoir"* by the late Charles L. Harrison, M. Am. Soc. C. E., and Silas H. Woodard, M. Am. Soc. C. E. The late R. Shirreffs, M. Am. Soc. C. E., in his discussion of that paper develops the following formula for the crown deflection of an arch dam.

$$D_c = \frac{CC_c \times P_1 \text{ (up-stream radius)}^2}{E \times t} \quad \text{.............(5)}$$

where $P_1 = P \times \frac{R_u}{R_m}$, and CC_c is a factor which takes the curved beam action into consideration and can be found directly from Fig. 6, which for the sake of ready reference, and on account of a typographical error in the original formula, is reproduced here from Fig. 22 in

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 89.

Mr. Shirreffs' discussion. E is the modulus of elasticity and t is the thickness. This formula and curve, however, do not take the initial stresses in the dam structure into consideration, and, therefore, before applying this formula for finding arch deflections due to the water load, we must determine how much of this load is carried by the arch due to initial stresses, because, only after deducting this part of the load does the remainder divide up between arch and cantilever action in the ordinary sense.

By initial stresses are meant stresses principally due to the weight of the structure and to the water pressure. Therefore, these stresses reach their maximum values at or near the foundation, and are zero at the crest. Thus far, they have not been much discussed, but they are very important, and should be taken into consideration when attempting to find the actual division of load between the arch and cantilever.

When the dam is completed, the lower layers of the dam body are in compression, due to the weight of the material above. When a body is compressed, the dimension in the direction of the compressive force becomes smaller, but in other directions the body swells, if free to move (lateral strain). The ratio of lateral to longitudinal strain is different for different materials. It is known to lie between $\frac{1}{3}$ and $\frac{1}{4}$ for bodies such as metals, but for concrete, which is not a very homogeneous substance, this ratio is about $\frac{1}{5}$ at the unit compression generally used in arch dam design. A. N. Talbot, M. Am. Soc. C. E., gives* values for this ratio, $\frac{1}{m}$, ranging from 0.1 to 0.18 at low loads, up to 0.25 at loads near the ultimate for 1 : 2 : 4 concrete 60 days old. A French commission on reinforced concrete found Poisson's ratio, $\frac{1}{m}$, for ordinary concrete to be about 0.16 when a force of 200 lb. per sq. in. was applied perpendicularly to the largest dimension of the concrete, and about 0.22 when applied at right angles to the largest dimension. Professor C. Bach, of Stuttgart, Germany, found for Poisson's ratio the value $\frac{1}{5.3}$ for 1 : 2 : 3 concrete 45 days old at a unit compression of about 180 lb. per sq. in.†

* *Bulletin No. 20*, University of Illinois.

† More details may be found in *Engineering News*, Vol. 68, p. 208.

A dam body often contains considerable quantities of large stones, and the presence of these stones has the effect of increasing the value of Poisson's ratio of the total mass. (Poisson's ratio for rock is about $\frac{1}{4}$.) Therefore the use of $\frac{1}{5}$ for this ratio for the total mass seems to be entirely justifiable. Any horizontal layer of material will have to sustain compression corresponding to the height of the masonry above it, and, therefore, will actually become shorter in a vertical direction and have a tendency to expand horizontally. If the abutments are unyielding, the arch may be prevented from actually becoming longer, in which case axial compression is introduced, the same as if water pressure acted on the structure. The value of this initial axial compression per square unit area is equal to the vertical compression multiplied by $\frac{1}{5}$ if the abutments are unyielding, and, as far as the final result is concerned, it does not make any difference whether they are absolutely unyielding or not.

If the specific gravity of the concrete for the dam is taken at 2.3 and the height of the dam at H, then the average vertical pressure can be expressed as $\frac{2.3\ H}{a}$, where a is the ratio of the total height of the dam to the height of a rectangular wall having the same sectional area and the same base. The ratio, a, is known as soon as the section is known, and in dam design the section must be more or less determined before final calculations can be made.

The dam section shown in Fig. 7 has an area of 9 668 sq. ft., a base width of 70 ft., and a height of 250 ft. The height of the masonry column causing the mean vertical pressure, therefore, is $\frac{9\ 668}{70} = 138$ ft., and $a = \frac{250}{138} = 1.81$, making the mean vertical compression on the foundation—in terms of head of water—equal $\frac{2.3\ H}{1.81} = 1.27\ H$, with no water pressure acting on the up-stream side.

The condition of reservoir full introduces an additional force—the radial water pressure—tending to compress the dam body in a direction perpendicular to the direction of the compressive force due to the weight of the body. At the bottom of the dam this force is equal to H, in case the water is standing to the crest of the dam. In this

case the radial water pressure tends to counteract the swelling of the concrete in an up-and-down-stream direction (due to the weight), thereby introducing additional initial axial compression.

The total resulting initial axial compression at the foundation of the section shown in Fig. 7 (in terms of head of water), therefore, is

$$\frac{1}{5}(1.27\,H + H) = 0.454 \times H \ldots\ldots\ldots\ldots\ldots (6)$$

where Poisson's ratio has been taken as $\frac{1}{5}$. Mr. H. Ballet, a French

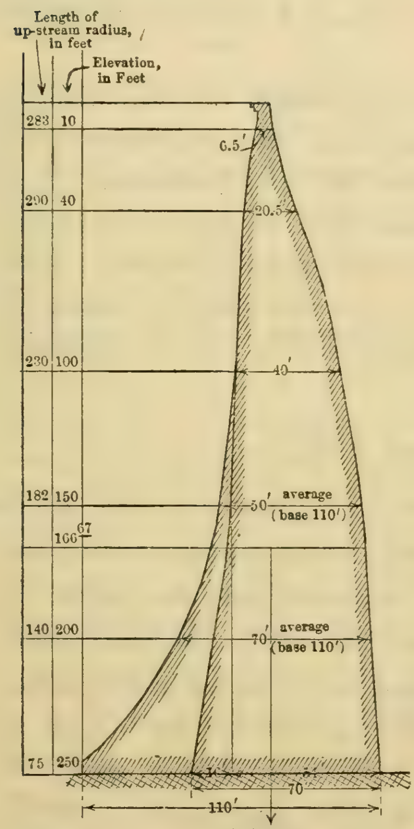


Fig. 7.

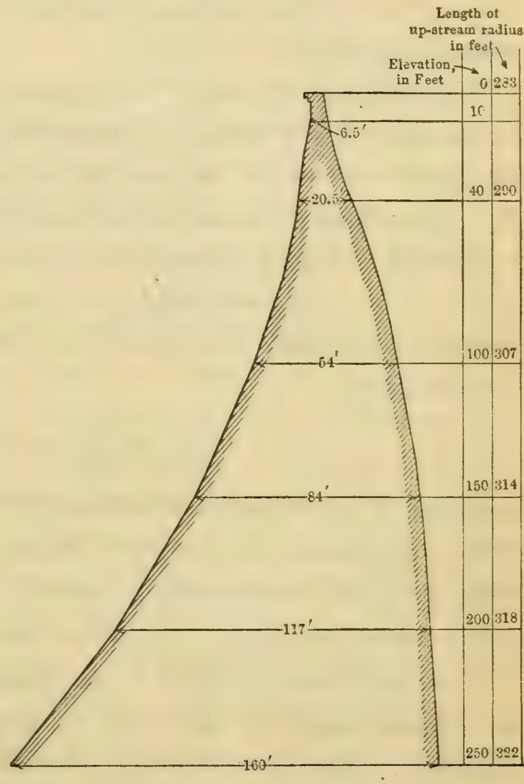


Fig. 8.

engineer, was probably the first to point out the necessity of taking Poisson's ratio into consideration when attempting to find the actual stresses in a dam body.*

The height of water, h, that this initial axial compression of 0.454 $\times$ H will resist without causing any shortening of the length of the arch at the bottom can be found by using Formula 1, thus

$$h = 0.454 \times H\frac{t}{R_u} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII, 1909, p. 51.

For the narrower section shown in Fig. 7, $t = 70$ ft. at the base, $R_u = 75$ ft. Substituting these values, it is seen that this section at the very bottom is able to carry,

$$h = 0.454 \times H \times \frac{70}{75} = 0.425 \times H,$$

or 42.5% of the total head of water, as an arch, before any shortening in the length of the arch occurs.

The initial axial compression holds in equilibrium the stresses due to 42.5% of the total head at the bottom, the remaining 57.5% of the load will divide between cantilever, arch, and curved-beam action, in the usual way.

By analyzing Formula 7 it is seen that by simply varying t and R_u, the designer has it within his power to make the arch and curved beam take more or less of the load by utilizing the initial stresses. If, in Fig. 7, the base thickness is increased from 70 ft. to 110 ft., and correspondingly at higher elevations, and R_u is kept at 75 ft., which is about as short as it is practicable to make it for the site for which the section is designed, then the initial stresses will be able to support at the foundation,

$$0.4 \times H \frac{110}{75} = 0.585 \times H,$$

or 58.5% of the total water pressure before any shortening in the length of the arch occurs, and before additional axial compression is introduced. When the arch becomes thick, however, compared with its length, the load is carried mostly on the curved beam; this will be discussed later.

The dam shown in Fig. 7 was designed with varying radii to keep the central angle of the arch as nearly constant as possible at all elevations. For comparison, a section is shown in Fig. 8, using the same unit compression (except where it is wider than a gravity section near the foundation) and the same up-stream face batter, but a single common center for both up-stream and down-stream faces. For this section the length of the up-stream radius is also variable, but it increases toward the bottom and reaches here a value of 322 ft.

The average vertical compression on the foundation for the section shown in Fig. 8 is $\frac{2.3}{2.2} H = 1.04 \times H$, and the corresponding total initial axial compression due to the lateral deformation is

$$\frac{1}{5}(1.04 \times H + H) = 0.41 \times H.$$

The height of water which this initial stress can resist, therefore, is equal to (see Formula 7)

$$0.41 \times H \frac{160}{322} = 0.20 \times H,$$

or 20% of the total head. Comparing this with 58.5%, or 42.2% for the constant-angle arch type (Fig. 7), it is easily seen that this latter type is much more effective in utilizing the initial stresses to support the water load than the ordinary arch having its faces struck from a single center.

If a gravity section is insisted on for the arch, but the central angle is kept as nearly constant as practicable, it will be possible for the gravity section to take up the greater part of the load, acting as an arch and curved beam toward the bottom. The factor of safety has thereby been increased several times with only a slight increase in material due to the smaller average radius used in the construction of the constant-angle arch.

It may be argued that shrinkage of the concrete while setting will compensate, to a large extent, for the axial expansion. This is true when the reservoir is empty and the concrete dry, but this condition is not of nearly as much interest or importance as that of reservoir full, for which condition it is not true. In this latter case the concrete is water-soaked, and in that state the length of the arch is at least equal to the original length before setting.* It has been shown in actual practice, however, that some concrete does shrink considerably, opening up contraction joints and cracks. One way of compensating for this, is to force grout into these contraction joints, at a time when the temperature is low and the reservoir empty, after having been filled once. In this way, the permanent set of the structure will also be compensated for, and the arch and cantilever will take their respective shares of the load when the reservoir is filled the next time, and each time thereafter.

Formula 5 has been used for finding the deflection curves, *A* and *B*, of the two sections, Fig. 7 (base 110 ft.) and Fig. 8. Formula 7 has been used for correcting these curves, *A* and *B*, to take the effect of lateral strain into consideration. These curves are shown in Fig. 9, and represent the deflections of the two arches, assuming that they

* See tests reported by Messrs. A. T. Goldbeck and A. H. White at the 1911 Convention of the American Society for Testing Materials.

are free to move at the foundation. They are plotted to show how evenly the deflection curve, *A,* slants from a maximum near the top (above Elevation 40 the two arches are identical, the "horns" being two tangents 40 ft. high) to nearly nothing at the bottom in the constant-angle arch type (Fig. 7), and how little the slant curve, *B,* amounts to in the ordinary type of arch dam (Fig. 8). These curves also show very plainly that from the common arch type much arch action toward the bottom cannot be expected; cantilever and beam action must take the load, because no such deflection as 0.2624 in. could be possible at the point where the arch is fastened to the rock foundation. The constant-angle arch type for this particular site, requiring only 0.0083 in. deflection, or 31.5 times less, to support the same load, will take most of the load on itself, acting as an arch.

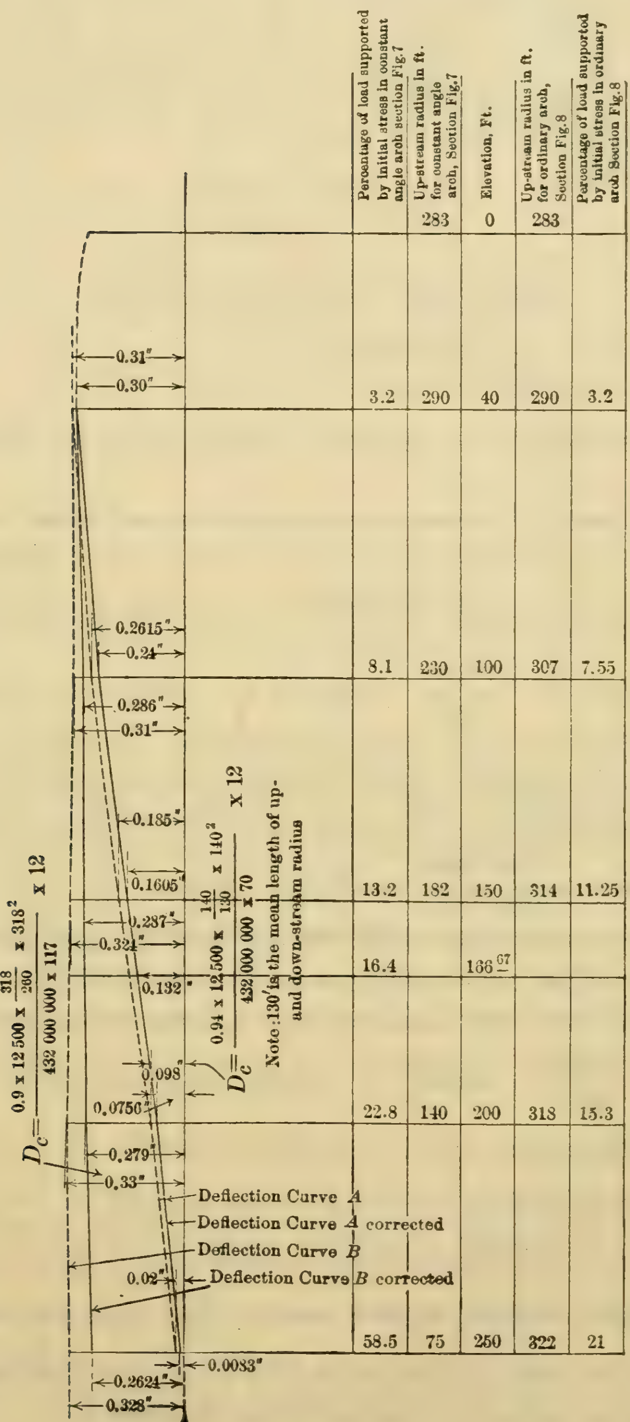
For dam sites where the abutments are close to one another toward the foundation, and where t is large compared with R_u, Formula 5 gives values for the crown deflection which are too large, even assuming that the dam is entirely free to move at the bottom. Though this formula considers the curved-beam action, it is at the same time understood that arch action is complete. However, where the arch is thick and the distance between the abutments is short, the arch becomes a wedge, and the horizontal curved beam takes the greater proportion of the load, because, acting in this manner, the support of the same load will require a smaller deflection.

The deflection in the middle of a beam 1 ft. wide, held at both ends, and uniformly loaded, is

$$D_b = \frac{P \times l^4}{E\,J \times 384} \quad \text{.......................} (5a)$$

The notations are the same as before, P being the water pressure, in pounds per square foot; l, the length of the span, in feet; E, the modulus of elasticity of concrete per square foot; and J, the moment of inertia.

Whenever Formula 5*a* gives smaller values than Formula 5, it is indicated that arch action is incomplete. The curved-beam action tends to introduce axial tension along the down-stream face in the middle, and along the up-stream face near the abutments, but the axial compression due to the partial arch action and lateral expansion (Poisson's ratio) will, or should much more than, compensate for this tendency. If it does not, the design should be changed.



Percentage of load supported by initial stress in constant angle arch section Fig.7
Up-stream radius in ft. for constant angle arch, Section Fig.7
Elevation, Ft.
Up-stream radius in ft. for ordinary arch, Section Fig.8
Percentage of load supported by initial stress in ordinary arch Section Fig.8
283
0
283
3.2
290
40
290
3.2
8.1
230
100
307
7.55
13.2
182
150
314
11.25
16.4
166 67/
22.8
140
200
318
15.3
58.5
75
250
322
21
0.31"
0.30"
0.2615"
0.24"
0.286"
0.31"
0.185"
0.1605"
0.287"
0.324"
0.132"
0.098"
0.0756"
0.279"
0.33"
0.02"
0.0083"
0.2624"
0.328"
$D_c = \frac{0.9 \times 12\,500 \times \frac{318}{280} \times 318^2}{432\,000\,000 \times 117} \times 12$
$D_c = \frac{0.94 \times 12\,500 \times \frac{140}{130} \times 140^2}{432\,000\,000 \times 70} \times 12$
Note: 130' is the mean length of up- and down-stream radius
Deflection Curve A
Deflection Curve A corrected
Deflection Curve B
Deflection Curve B corrected

FIG. 9.

From Curve A the deflection of the arch, Fig. 7, at the one-third point can be ascertained directly, and is found to be 0.132 in. If the cantilever, 250 ft. high and 1 ft. wide, were actually forced to deflect 0.132 in. at this point (Elevation 166.67 ft.) a force, F, would be required, which can be found as follows (F is concentrated at the one-third point):

$$D_c = \frac{8 \times F \times l^3}{E \times h^3} \qquad (8)$$

$$D_c = \frac{0.132}{12} = \frac{8 \times F \times 83.33^3}{432\,000\,000 \times 110^3},$$

$$F = \frac{759\,000\,000}{555} = 1\,368\,000 \text{ lb.}$$

(Where no explanation is given, formulas are taken from standard handbooks.)

The cantilever will deflect the same as the arch when thus loaded.

The total water load on a vertical slice of the dam, 1 ft. wide and 250 ft. high, is $250 \frac{0. + 15\,625}{2} = 1\,953\,125$ lb. The initial stress supports $1\,953\,125 \times \frac{16.4}{100} = 320\,312$ lb. before any deflection takes place, therefore the load causing a deflection of 0.132 in. of the combined arch and cantilever must be equal to $1\,368\,000 + 1\,953\,125 - 320\,312 = 3\,000\,813$ lb. The proportion of this amount taken by the cantilever will be $\frac{1\,368\,000}{3\,000\,813} = 45.5$ per cent.

Now, the actual load to be divided between cantilever and arch is not 3 000 813 lb. per running foot, but only $1\,953\,123 - 320\,312 = 1\,632\,813$ lb. Of this amount the cantilever carries 45.5%, or $1\,632\,813 \times \frac{45.5}{100} = 742\,930$ lb., concentrated at the one-third point, making the actual deflection at this point $0.132 \times \frac{55}{100} = 0.073$ in.

The bending moment due to this force is equal to

$$742\,930 \times 83.33 = 61\,908\,357 \text{ ft-lb.}$$

The section modulus of the base $= \frac{110^2}{6} = 2\,011$, and therefore the compressive stress on the foundation at the toe, due to the bending action of the water load on the cantilever, is equal to

$$\frac{\text{Bending moment}}{\text{Section modulus}} = \frac{61\,908\,357}{2\,011} = 30\,780 \text{ lb. per sq. ft.} \qquad (9)$$

The total compression on the foundation at the toe will be this compression added to that due to the weight of the structure, which amounts to approximately 16 200 lb. per sq. ft. at the toe, making the total compression approximately 47 000 lb. per sq. ft.

If a base length of 70 ft. is chosen, the arch would take a greater percentage of the load and the curved beam a smaller, leaving the same or less for the cantilever, but, owing to the smaller section modulus of the 70-ft. base, the compression at the toe would be somewhat higher than 30 780 lb. per sq. ft., and the compression due to the weight of the structure would be much higher than 16 200 lb. per sq. ft., so that the sum of the two would be considerably more than 47 000 lb. per sq. ft. Although within the safe limit, the resulting vertical compression would be out of proportion to the 36 000 lb. per sq. ft. (and less) axial compression used when calculating t from Formula 1.

The dam section with the 110-ft. base contains only 4% more material than the dam with the 70-ft. base (Fig. 7), as the addition is not made as a portion of a circular ring, but in the shape of a spherical triangle, as indicated in Figs. 4 and 5. Any intermediate base length between the two limits given in Fig. 7 could be accepted for a dam built on this particular site. (This is shown approximately in Fig. 3, but there the tangents are not indicated.) The two stresses, the 36 000 lb. per sq. ft. average axial compression, and the maximum 47 000 lb. per sq. ft. vertical compression, are acting in planes perpendicular to each other, and therefore tend to support each other. Although they are low, the resulting section (Fig. 7) appears unusually slender on account of the economical distribution of the material.

This method of calculating the vertical stress on the foundation is correct only so long as no tension exists at the heel, or, if tension exists, so long as this tension is properly taken care of. For the constant-angle arch, where the cantilever takes the smaller proportion of the load, there will seldom be occasion for tension along the up-stream face, and there will perhaps never be enough tension to demand consideration. The accuracy of the result obtained from the use of Formula 8 depends to some extent on the face slopes, especially that of the down-stream face. The error, however, is generally such as to compensate for that made in not considering that the width of the

vertical cantilever, which is 1 ft. at the up-stream face, is less at the down-stream face. The short-cut method explained above for finding the division of the water load between cantilever and arch action and that for finding the total maximum foundation pressure, cannot be used for dams having a crown deflection curve similar to Line *B*, as this line does not show a maximum deflection near the crest and a zero deflection at the foundation. The deflection curve, *A*, answers these conditions closely enough for this purpose.

Thus far, only the middle or highest dam section has been considered, as we have been mostly interested in knowing the maximum stresses in the structure, which stresses generally occur (in high dams at least) at the toe, with reservoir full. Some modifications may be necessary near the abutments where the cantilevers, unless very short, do not support their share of the load, most of which is thrown over to the horizontal curved beam and arch, increasing the axial compression on the down-stream face from the abutments to somewhere beyond the points of contrary flexure. These points are located on both sides of the crown, and can be found from,

$$\cos. \theta_0 = \frac{\sin. \theta}{\theta} \qquad (10)$$

as shown by Fig. 1. Reference should also be made to the discussion on "Lake Cheesman Dam and Reservoir".* This higher axial compression will not be entirely local, but will be transmitted from abutment to abutment through the middle of the dam, making the axial compression in the middle higher than that due to the proportionate water load carried by arch action; although still lower than the value obtained from Formula 1, assuming the whole load to be carried by the arch. The actual value may be found by drawing a graphical stress diagram, such as used for finding stresses in arch ribs for concrete bridges, etc., keeping in mind that, in the case of a dam, the directions of the forces are radial. From information as to the distribution of the axial stresses thus obtained, a new value for the deflection at the one-third point can be ascertained, and some readjustment between arch and cantilever action can be made. As long as the unit compression used is not much greater than 50 000 lb. per sq. ft., this refinement seems to be unnecessary, and hardly worth the additional labor.

* *Transactions*, Am. Soc. C. E., Vol, LIII, p. 167.

In Formula 1 only average stresses have been considered in determining the thickness of each individual arch slice. The maximum axial stresses should also be investigated. These exist along the down-stream face, and are found from the formula,

$$Q\ (\text{max.}) = q \times \frac{2\ R_u}{R_u + R_d} \qquad \text{(11)}$$

Formula 11, however, does not give correct results toward the foundation, where the arch is thick relative to the length of the up-stream radius, and where the span is short. The proportion of the load carried by the arch in such a case is supported more by the curved-beam than by ordinary arch action. This will cause some difference in the value of Q (max.) and q (min.) (as found from Formula 11), adding to Q (max.) at and toward the abutments, and subtracting from it in the middle portion between the points of contra-flexure on the curved beam. In high dams, Q (max.) will ordinarily be lower than the vertical compression at the toe, therefore this vertical pressure is still the most important to investigate.

The influence of Poisson's ratio tends to equalize Q (max.) and q (min.) in dam sections having up-stream faces of steeper slope than their down-stream faces. In such sections the vertical pressure due to the weight of material above is greatest along the up-stream face, and therefore the initial axial compression is also greatest. It is fair to assume that this condition of relieving Q (max.) and adding to q (min.) also tends to improve the water-tightness of the dam.

In all straight gravity dams built across narrow canyons, horizontal tension exists along the down-stream face in the middle, and along the up-stream face near the abutments, at least toward the foundation. This should be very plain when it is considered that any beam fixed at both ends and uniformly loaded will support four times as much load as a cantilever of the same length sustaining the same water load (nothing at the top and a maximum at the bottom). In other words, whenever the beam is four times longer than the cantilever, it will support half of the total load, and whenever this ratio is less than four, the horizontal beam will support most of the load. The ordinary gravity design does not consider this beam action, although, when the dam is built in a fairly narrow canyon, the greatest portion of the load toward the foundation is actually carried on the

horizontal beam, and not on the cantilever. Though adding materially to the stability of the dam (as long as the horizontal tension introduced by this beam action is not greater than the breaking point, and as long as the expansion joints, if any, are placed at or near the points of contra-flexure), the foundation pressure at the toe is at the same time much relieved, a very welcome feature, especially in connection with high dams, and surely this feature should not be left out of consideration when calculating the factor of safety.

Now, if the horizontal beam is curved, horizontal axial compression takes place over the entire section, and the greater the curvature (that is, the smaller the up-stream radius) the more load will be taken by the arch, and the less remains to cause horizontal axial tension at any point of the dam faces, due to beam action. The resultant axial compression from arch action and lateral expansion will in general more than compensate for this tension. Lateral expansion due to the weight of the structure exists, of course, whether the dam is straight in plan or curved, but this alone will seldom be sufficient to compensate for the horizontal tension due to beam action in a straight gravity dam across a narrow canyon. The curvature must be introduced in order to be sure that no tension exists in this horizontal beam. For a dam 250 ft. high, the bottom width of the canyon would have to be well toward ½ mile before a gravity dam would act simply as a gravity section toward the bottom, and before the influence of the horizontal beam action would be negligible, unless it should have failed in tension first. (Near the top, the horizontal beam would have no practical influence.) It would seem logical, therefore, to provide even quite long dams with a curvature sufficient to take care of the horizontal tension. Such dams would not be true arch dams, but the slight curvature would increase the factor of safety by eliminating the tension in the horizontal beam; and combined beam, cantilever, and some arch action would support the load.

Whenever a load is supported on a beam, or on a cantilever, shearing stresses are introduced. These shearing stresses reach their maximum values at the foundation and at the abutments, and should be investigated in order to be sure that they are within safe limits. In the case of the dam section shown in Fig. 7, base 70 ft., it can easily be shown that, even should the shear on the lower 50 ft. of the dam

correspond to the full water pressure, this stress would be entirely within the safe limit, amounting to approximately 4 000 lb. per sq. ft. where the dam joins the hillside and foundation. Along this joint the maximum unit compression generally exists. This compression is so much larger than the shear that actual shear cannot take place, as friction alone will prevent any tendency to sliding at the abutments. Crushing would have to take place before the actual sliding of any element could occur. A fair approximation of the magnitude of the actual shear forces along the joint between the dam foundation and abutments can be obtained in the following manner: Formulas 5 and 5*a* are both applied in order to find in what proportion the load is divided between arch action and curved-beam action toward the bottom. Suppose Formula 5*a* gives, say, half as much deflection as Formula 5 for the same load; then it is indicated that the curved beam carries twice as much load as the arch. In reference to this the factor, CC_c, in Formula 5 should be left out, as we are working close to the foundation, where the arch is short and thick, and where it is therefore necessary to use both Formulas 5 and 5*a*.

From Formula 7 the percentage of the total load carried by the initial axial stresses can be found, and from Formula 8, etc., the percentage of the remaining load (due to head, $H - h$) taken by the cantilever, can be determined. In the case of the dam shown in Fig. 7, this percentage was found to be 45.5, and there is, therefore, 54.5% (of $H - h$) left for the arch and curved beam. Out of this the arch takes $\frac{1}{3} \times 54.5\%$, and the curved beam $\frac{2}{3} \times 54.5\%$ if Formula 5*a* gives half as much deflection as Formula 5. The shear is caused by a force equal to $45.5 + \frac{2}{3} \times 54.5\%$ of the load that remains after that carried by the initial axial stresses (Poisson's ratio) has been subtracted from the total load. Another place where shear action exists is in a vertical plane at the foundation along the toe when the reservoir is full of water. The tipping in a down-stream direction of the loaded cantilever deforms the toe and forces it to make a depression in the foundation, and, as the rock foundation extends outside the concrete toe, shear will take place where the end of the toe tries to deform the foundation. This circumstance introduces additional foundation

pressure at the toe, but this increase is already included in the value as found from Formula 9, at least approximately.*

Before concluding the general discussion, the action of changes in temperature should be considered. As the ends of the arch are fixed to the abutments, the shortening or lengthening of the arch due to temperature changes either causes cracks to develop or forces the crown back and forth. In the constant-angle arch type the average crown deflection for the same amount of decrease or increase in length of arch is only about half of that required by the common arch type (see Curves *A* and *B*, Fig. 9), and the tension necessary to cause this deflection or pulling back of the cantilever, therefore, may not exceed the ultimate strength of the concrete, in which case no cracks would develop. In any event, cracks are not as likely to occur as in the common arch type of dam.

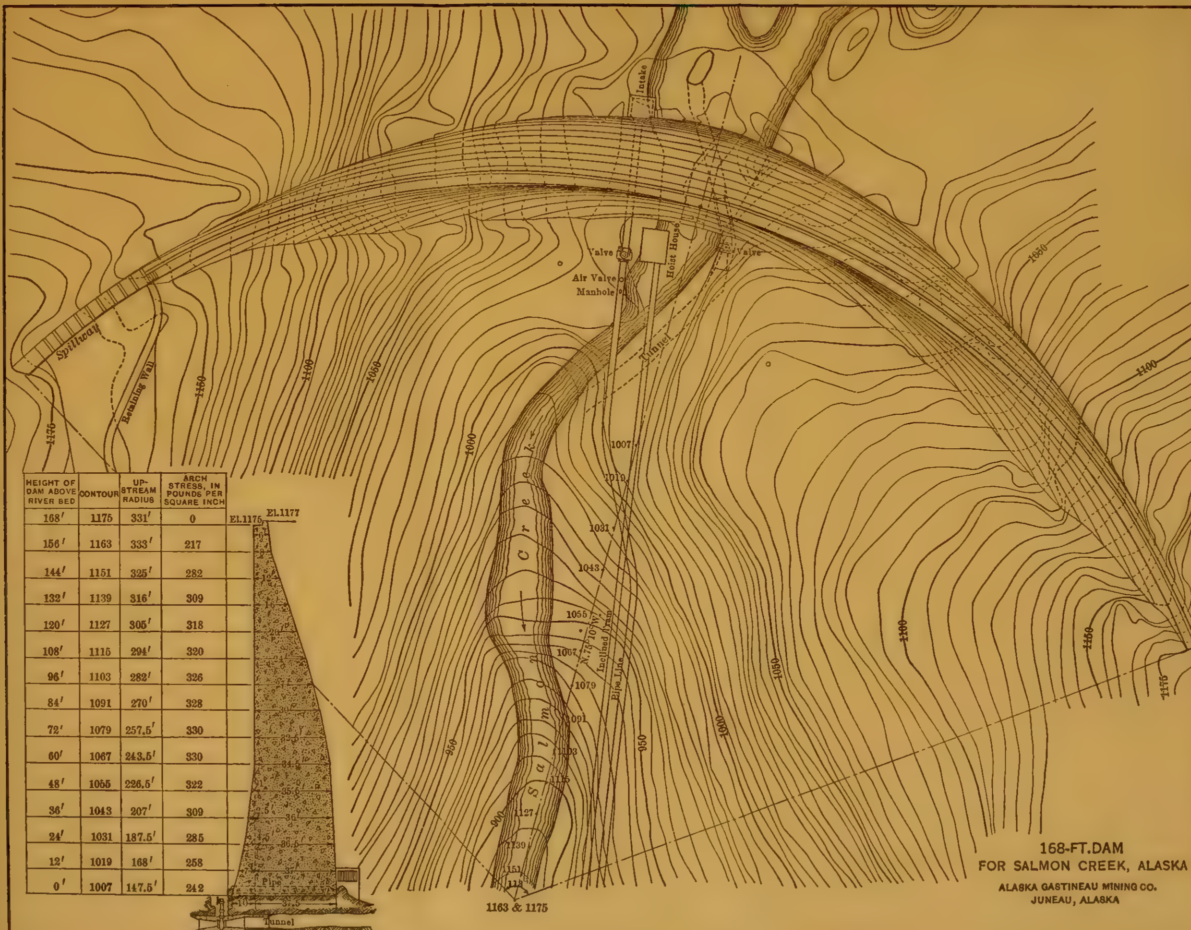
Practical Examples of Arch Dams.

Plates IX and X show the plan and section of the Salmon Creek Dam, 168 ft. high above the river surface, containing 52 000 cu. yd. of concrete, having 1.25 bbl. of cement and a small percentage of lime per cubic yard. The contours represent the actual condition of the site after excavation, and it is seen that the sides of the canyon form an unusually regular V. The crest width of the dam is approximately 550 ft., measured in a straight line, and the arch at this elevation subtends an angle of 113 degrees. Plate IX shows the location of the centers for various arch slices, 12 ft. apart in elevation, and the length of the corresponding up-stream radii and unit axial stresses are given in the table on Plate X. To provide better accommodation for the spillway, the curve for the top 12 ft. of the dam was struck from the same center, therefore the warping of the faces commences 12 ft. below the crest, and continues down to the foundation. This warping is so uniform that one who does not know that the centers are moved constantly in an up-stream direction toward lower elevations does not notice it. The carpenter gets his points about every 10 ft. apart, and it makes no difference to him whether he builds up the

---

*The result from Formula 9 depends on Formula 8, and in Formula 8 the deflection due to the bending only is considered, not the additional deflection due to the horizontal shear forces accompanying the bending action, which amount to 10% (average), therefore the proportion of load taken by the cantilever, as given on page 1302, is really 10% too high, but this makes the resulting foundation pressure at the toe, found from Formula 9, more nearly correct on account of the action of the vertical shear at the toe, and this is the information that concerns the designer most.

PLATE IX.
PAPERS, AM. SOC. C. E.
MAY, 1914.
JORGENSEN ON
THE CONSTANT-ANGLE ARCH DAM.



HEIGHT OF DAM ABOVE RIVER BED	CONTOUR	UP-STREAM RADIUS	ARCH STRESS, IN POUNDS PER SQUARE INCH
168′	1175	331′	0
156′	1163	333′	217
144′	1151	325′	282
132′	1139	316′	309
120′	1127	305′	318
108′	1115	294′	320
96′	1103	282′	326
84′	1091	270′	328
72′	1079	257.5′	330
60′	1067	243.5′	330
48′	1055	226.5′	322
36′	1043	207′	309
24′	1031	187.5′	285
12′	1019	168′	258
0′	1007	147.5′	242

168-FT. DAM
FOR SALMON CREEK, ALASKA
ALASKA GASTINEAU MINING CO.
JUNEAU, ALASKA

face of a cylinder or an inverted cone (approximately). The surveyor, however, has to be more careful than with the layout of an ordinary arch, as, in the present case, there are more calculations to be made and to be followed.

The table on Plate X shows that the length of the longest up-stream radius is 333 ft., and the length of the shortest 147.5 ft., the ratio between the two being $\frac{333}{147.5} = 2.26$.

Had the length of the up-stream radius been kept constant, the thickness of the dam at the bottom would have had to be increased 2.26 times for the same axial stresses. Relative to this it should be noted that the arch stresses in the table assume the arch to take the total load, but in reality the stresses are somewhat smaller, as the cantilever takes part of the load. The triangular piece, 10 ft. wide at the bottom (which is added to the lower part of the dam on the down-stream side for the purpose of stiffening the cantilever where it is highest), is not considered in the table giving the arch stresses.

To have kept the enclosed angle (113°) constant at all elevations would have necessitated a greater ratio than 2.26 between the length of the two up-stream radii already referred to. Had this ratio been increased, the structure would have been overhanging in places, and therefore this increase could not be made. This simply shows that it is not always possible to make theory and practice coincide exactly.

To have kept the subtended angle constant in this case would have necessitated a greater bottom width of the site, other conditions remaining the same, but, of course, the dam has to be fitted to the site, and not *vice versa.* This dam creates a reservoir having a capacity of 826 000 000 cu. ft. The drainage area is only 7.5 sq. miles, but the precipitation makes up for it, being more than 100 in. per year.

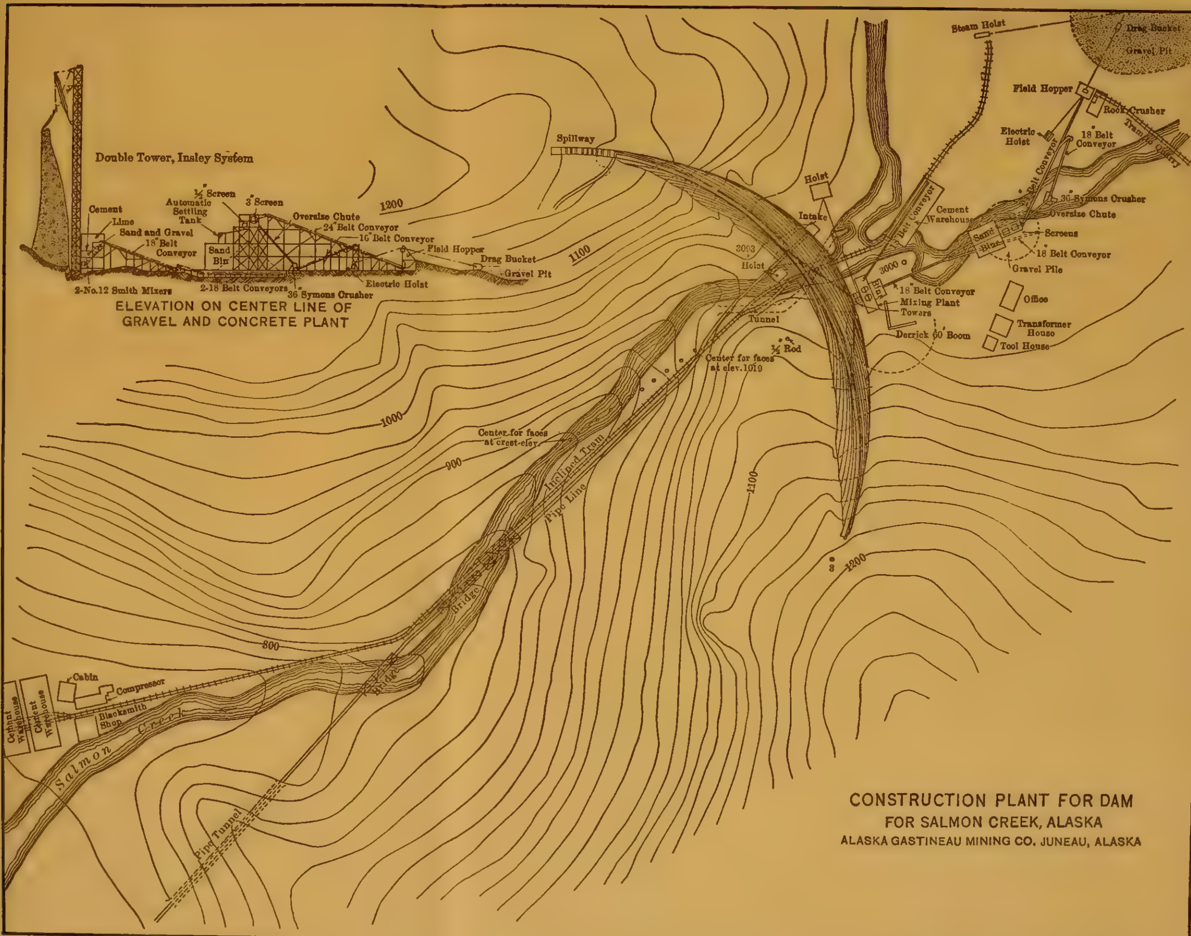
Plate X shows a plan and section of the construction plant, and is self-explanatory. This construction plant, which proved to be the best kind that could be used for this particular place, was laid out under the supervision of Mr. H. L. Wollenberg, who was Chief Engineer of the Alaska Gastineau Mining Company, Juneau, Alaska, the owners, and had charge of this and other work. F. G. Baum, Assoc. M. Am. Soc. C. E., was Consulting Engineer for the Mining Company. The original design of the dam was made by the writer. Plates IX and X, however, were made by the Gastineau Mining Company.

F. C. Herrmann, M. Am. Soc. C. E., Chief Engineer of the Spring Valley Water Company, San Francisco, A. P. Davis, M. Am. Soc. C. E., Chief Engineer of United States Reclamation Service, and the writer visited and reported on the project at various times, and Mr. Herrmann wrote an extensive report thereon.

Figs. 10 to 13, inclusive, show different construction features, thus: The double tower with the two hoppers from which the two concrete chutes distribute the concrete over the dam. These chutes are supported on triangular wooden towers, easily removable. Two $\frac{3}{4}$-cu. yd. Smith mixers were used, and the progress was at the rate of about 400 cu. yd. daily. Work was shut down for the winter about November 1st, 1913, at which time the dam was half completed. It is expected to have it finished by August 1st, 1914. Indications are that the unit cost will be $7.50, or slightly less, including everything. The electric power plant, depending on the reservoir for its steady supply, is now in operation. Mr. D. C. Jackling is Vice-President, in charge of the operations of the Gastineau Mining Company, and Mr. B. L. Thane is Manager, with offices in Juneau, Alaska.

Plate XI shows the constant-angle arch principle applied on the Lake Spaulding Dam. This dam is part of the South Yuba development, undertaken and owned by the Pacific Gas and Electric Company, San Francisco, Cal.* The lower portion of this dam (60 ft.) is provided with a gravity section for a 260-ft. head, and is arched in plan, the length of the up-stream radius being 600 ft. up to Elevation 4 628. When this elevation was reached (less at the down-stream face, as shown on Plate XI) work was shut down for the winter, 1912-13. During the winter the original plans were changed, and in the following summer the dam was continued to Elevation 4 825, according to the plans on Plate XI. At Elevation 4 628 the length of the up-stream radius was changed from 600 ft. to 250 ft. and kept at this length up to Elevation 4 675. From this level up to the crest, the length of the up-stream radius increases so as to keep the subtended central angle as constant as possible, as shown by the table on Plate XI. This subtended angle is not as large as could be desired, but is as great as the

*This development has been described in detail in articles in *Engineering News*, *Engineering Record*, and others during 1913. Probably the most complete and accurate article on the subject is that written by R. W. Van Norden, M. Am. Soc. C. E., in the *Journal of Electricity, Power and Gas*, San Francisco, December 13th, 1913.



CONSTRUCTION PLANT FOR DAM
FOR SALMON CREEK, ALASKA
ALASKA GASTINEAU MINING CO. JUNEAU, ALASKA

FIG. 10.—RESERVOIR SITE ON SALMON CREEK, ALASKA. LOOKING UP STREAM FROM THE DAM. GRAVEL PITS IN THE FOREGROUND.

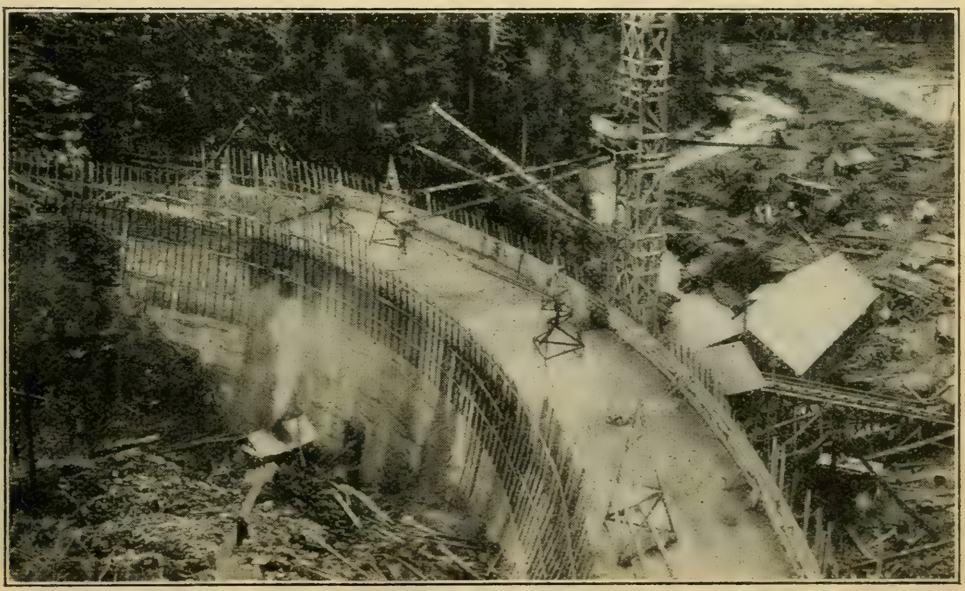


FIG. 11.—SALMON CREEK DAM, SHOWING CONSTRUCTION TOWER AND FORM WORK.

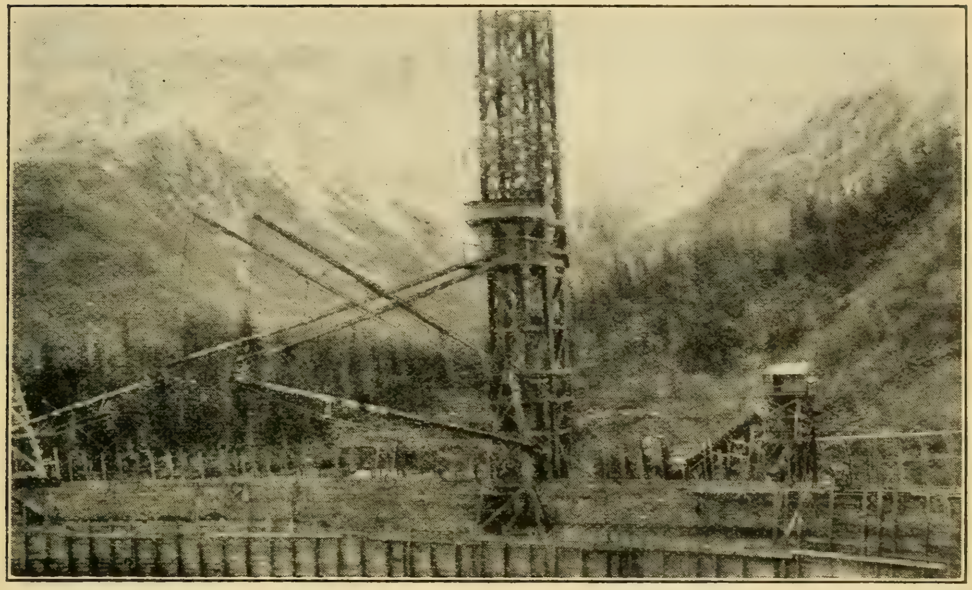


FIG. 12.—SALMON CREEK DAM. DETAILS OF TOWER AND DISTRIBUTING SYSTEM FOR CONCRETE.

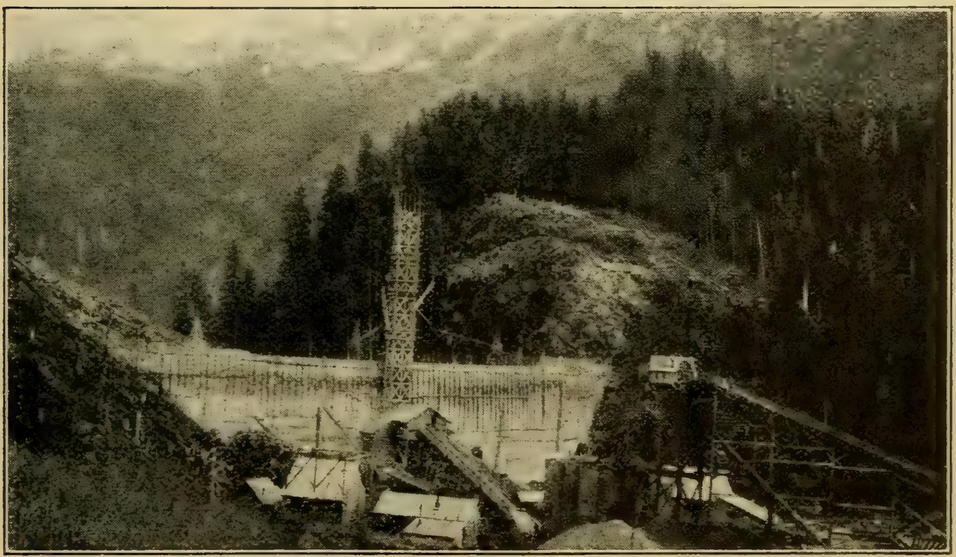


FIG. 13.—SALMON CREEK DAM. CONSTRUCTION PLANT.

site would permit, considering that the ultimate proposed crest elevation is to be at Elevation 4 905.

The Lake Spaulding Dam is provided with an inspection tunnel, a drainage system, and contraction joints, which are usual features in dams of large proportions. The details of these features are shown on Plate XI. The section of the arch above Elevation 4 660 is of such dimensions that it will stand an extension of 35 ft. in height above the present crest elevation (4 825) without any addition to its thickness. The maximum arch stress (q in Formula 1) will exist at Elevation 4 775 with the water level at Elevation 4 860, or 260 ft. above the river bed, and will amount to 23.8 tons. It is fairly constant over the greatest portion of the structure, as can be seen from the table on Plate XI.

When the time comes to extend the crest of the dam to Elevation 4 905, 305 ft. above the river bed, a slab of concrete must be added to the down-stream face, and, in order to effect a good bond between the present dam and the new slab, the down-stream face of the present dam has been stepped off and a sufficient number of iron rods have been left protruding several feet to grip the new slab and hold it in place.

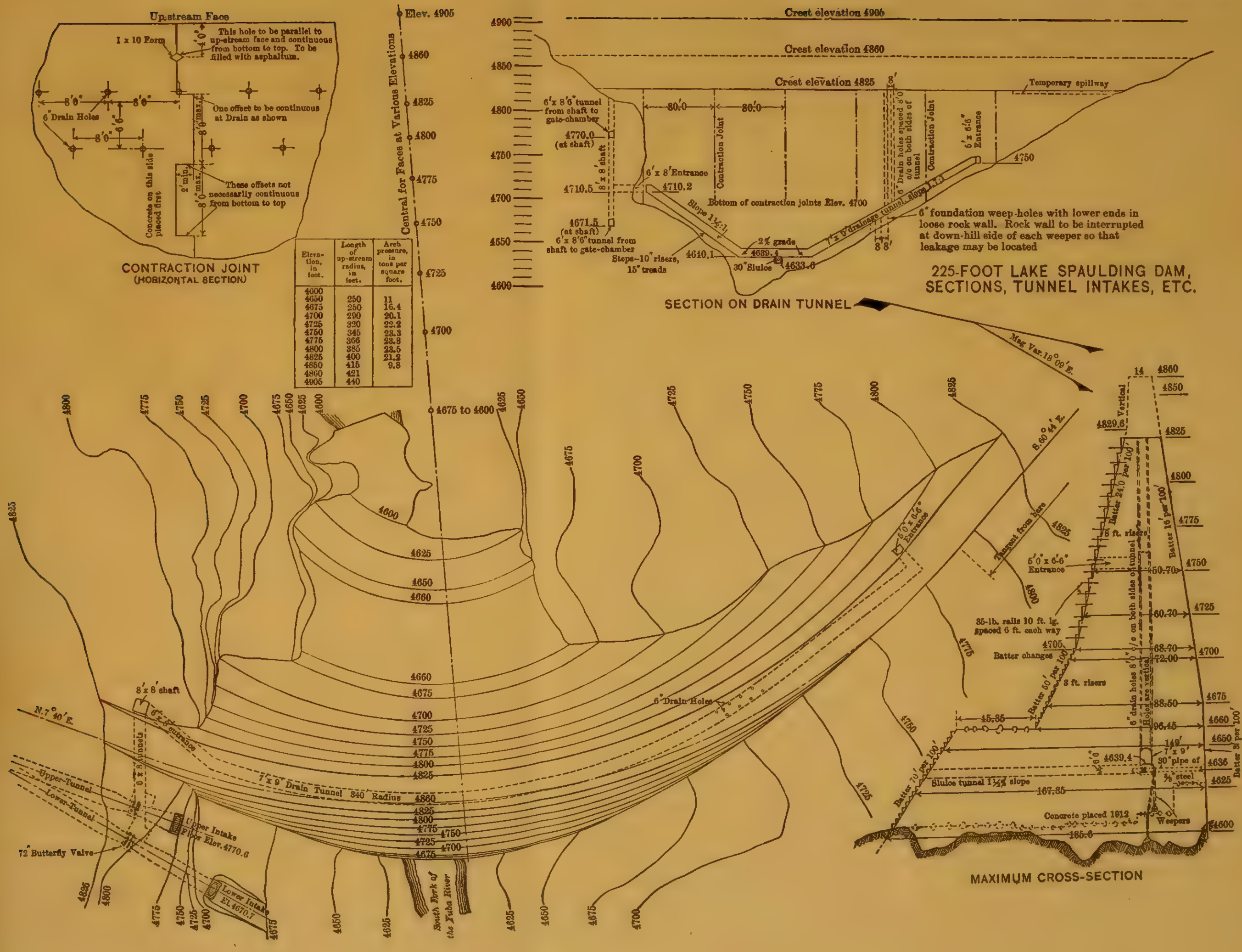
The Lake Spaulding Dam was constructed of a wet mixture of gravel concrete. Toward the top this mixture was richer than at the bottom, and contained from 1.17 to 1.25 bbl. of cement per cubic yard. Test specimens were broken every day, and the results of these tests were used as a guide in fixing the exact proportion of the mix. It was required that the breaking strength of the test specimens (8 in. in diameter) should be at the rate of 400 lb. per sq. in. when 7 days old; and 900 lb. per sq. in. when 28 days old. Toward the lower elevations, where the arch was thick, the mixture was somewhat leaner. The concrete was mixed in four 1-cu. yd. Smith mixers on the hillside above the dam, and, after being turned in the mixers for 1½ min., it was run by gravity in flumes to different portions of the dam, as shown on Figs. 14 and 15. After the dam had risen to a certain height the concrete could flow no more by gravity to portions farthest away, and a 30-in. belt-conveyor system was installed. With this system, all portions of the dam could be reached. There were also two cableways, to transport lumber and other materials to the dam. During the latter part of 1913 there was

very little water in the reservoir, the outside temperature was low, and the chemical heat was practically out of the dam body. This had the effect of opening up the contraction joints (which are 80 ft. apart) about $\frac{1}{8}$ in. In addition to this, cracks about $\frac{1}{16}$ in. wide, appeared midway between each contraction joint. These were all along radial lines, the same as the contraction joints. When the water in the reservoir rose to the crest, early in February, 1914, these cracks closed.

The outlet works from the reservoir consist of two intake tunnels (shown on Plate XI), concrete lined, and of a finished diameter of 8 ft. 8 in. One intake is at Elevation 4 670; the other is 100 ft. above. Each intake is provided with a 72-in. butterfly valve. The upper intake slopes downward about 48° until it meets the lower tunnel, this slope starting a few feet back of the upper butterfly valve. About 1 000 ft. down stream the single pressure tunnel ends in an adit, and is there provided with a second butterfly valve and also with two pressure reducers. Later, it is intended to install a 5 000-kw. turbine and let this act as a pressure reducer by utilizing whatever head there may be in the reservoir. From this point the 350 sec-ft. of water will flow by gravity toward the power-houses below, of which one is built, and four more are projected. After the water has left the lowest power-house, it will be used for irrigation.

The actual construction work was started under the direction of the late James H. Wise, Assoc. M. Am. Soc. C. E. It was continued and completed to date with F. G. Baum, Assoc. M. Am. Soc. C. E., as Chief Engineer. John R. Freeman, A. P. Davis, and H. F. A. Schussler, Members, Am. Soc. C. E., and others acted as Consulting Engineers at different times. Mr. Freeman, working in conjunction with Mr. Davis, made an extensive report, and many of the details suggested by these gentlemen are incorporated in the construction of the dam. Mr. Davis visited the site several times during the construction period. In the actual design as used, the writer's arch theory, and the shape of the up-stream face suggested in designs submitted by him, have been followed, except near the foundation, where an ordinary conical face was substituted below Elevation 4 675, instead of continuing the inverted cone (approximately) above.

Mr. John A. Britton was General Manager and Vice-President of the Company, H. C. Vensano, Assoc. M. Am. Soc. C. E., Civil Engineer. R. G. Clifford, Assoc. M. Am. Soc. C. E., Designing Engineer



Elevation, in feet.	Length of up-stream radius, in feet.	Arch pressure, in tons per square foot.
4600		
4650	250	11
4675	250	16.4
4700	290	20.1
4725	320	22.2
4750	345	23.3
4775	366	23.8
4800	385	23.5
4825	400	21.2
4850	415	9.8
4860	421	
4905	440	

225-FOOT LAKE SPAULDING DAM, SECTIONS, TUNNEL INTAKES, ETC.

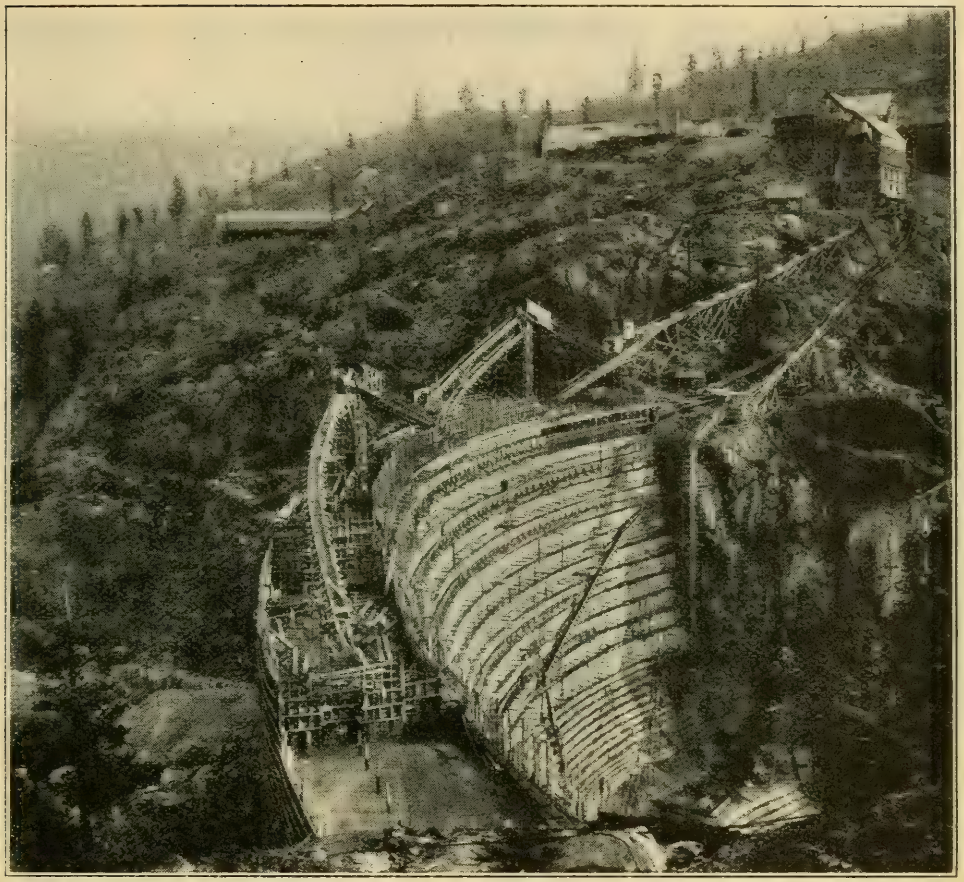


FIG. 14.—LAKE SPAULDING DAM. MIXING HOUSE, DISTRIBUTING FLUMES, AND BELT CONVEYORS. THE METHOD OF CONSTRUCTING THE EXPANSION JOINTS CAN BE SEEN.

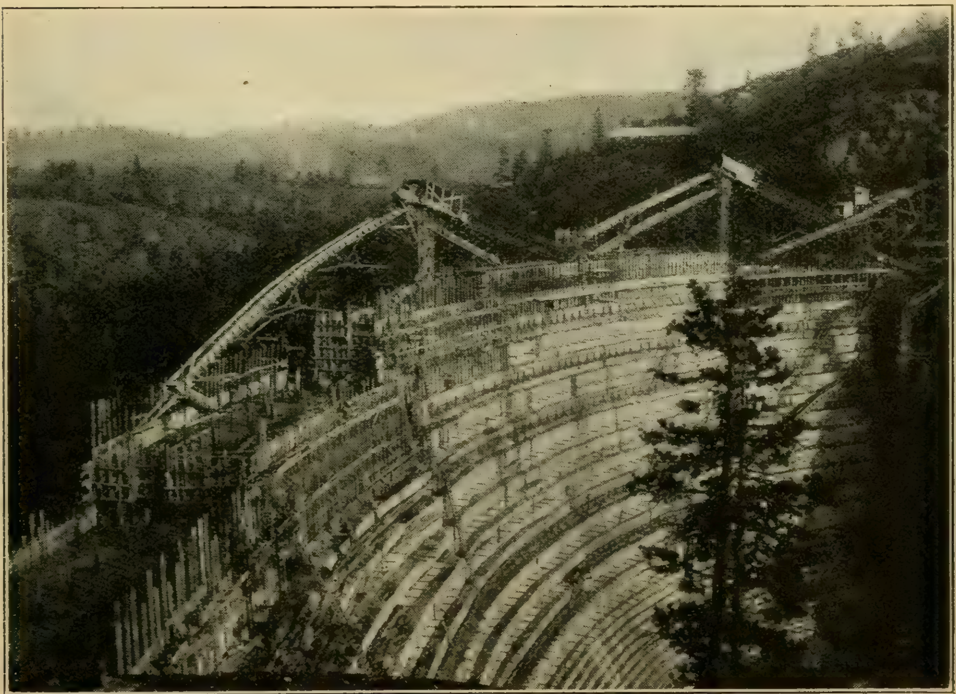


FIG. 15.—DOWN-STREAM VIEW OF LAKE SPAULDING DAM, SHOWING THE STEPPING-OFF AND IRON RODS FOR BOND.

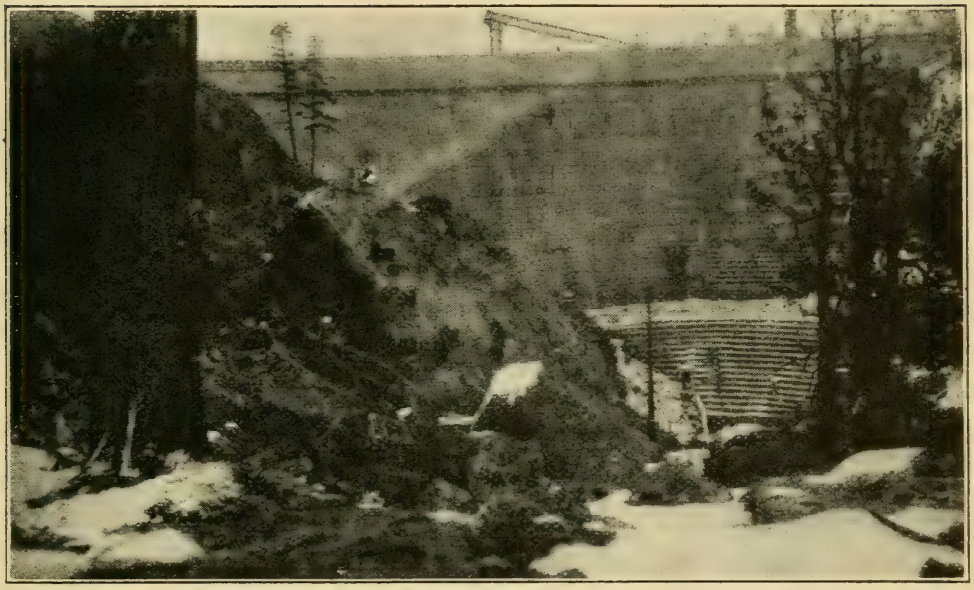


Fig. 16.—Down-stream Face of Lake Spaulding Dam, 225 Feet High Above River Bed.

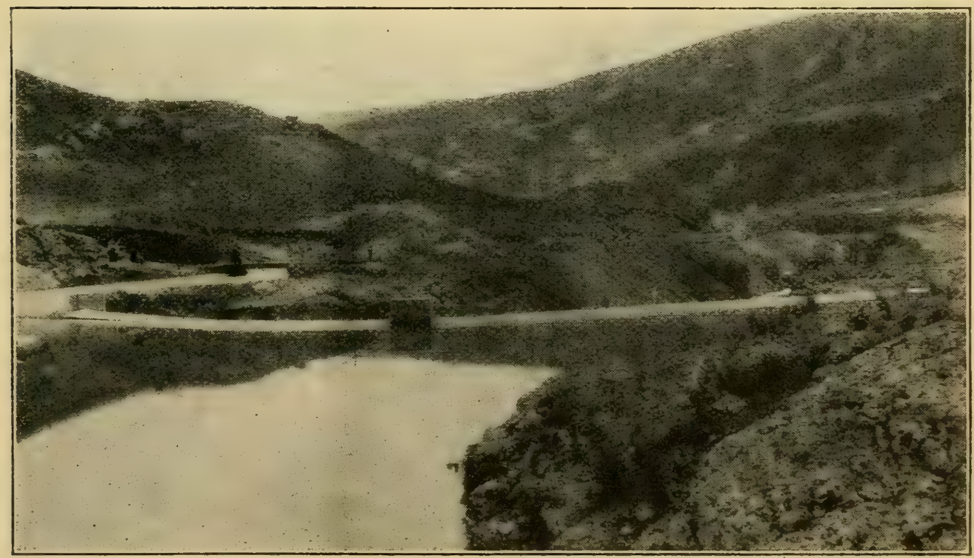


Fig. 17.—A Small Overflow Dam. Showing Obstruction in the Middle to Facilitate the Admission of Air Behind the Sheet of Falling Water.

and Resident Engineer when the dam was started. Mr. P. Magerstadt was Resident Engineer during 1913, and Messrs. Duncanson and Harrelson superintended the construction.

An arch dam 30 m. (98.4 ft.) high, with a base width of 4 m. (13.12 ft.) and a crest width of 1 m. (3.28 ft.), having an up-stream radius of 20 m. (65.6 ft.) at the bottom and 35 m. (114.8 ft.) at the crest, has been designed and built by H. F. Cameron, M. Am. Soc. C. E., Division Engineer of the Bureau of Public Works, Manila, P. I., to store water for domestic use.*

Fig. 17 shows a small diverting overflow dam designed on the constant-angle arch principle. It is built at an elevation of 13 000 ft. above sea level, in the Peruvian Andes, and is part of a hydraulic development by the Cerro de Pasco Mining Company. Messrs. F. G. Baum and Company, of San Francisco, were the engineers, and Mr. H. L. Wilcox was the Superintendent. This dam is designed to take care of a maximum overflow of 1 500 sec-ft., and is 62 ft. high, from bed-rock to crest, but only 24 ft. high above the river bed. The length of the up-stream radius at the crest is 36 ft., and at the river bed, 18 ft.

* *Engineering Record*, August 23d, 1913, p. 203.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SUBAQUEOUS HIGHWAY TUNNELS.

By George Duncan Snyder, M. Am. Soc. C. E.

To be Presented September 16th, 1914.

Synopsis.

The object of this paper is to call attention to the use of tunnels as a means of crossing rivers or bodies of water by highways, particularly when the waters are navigable, and to discuss the conditions where tunnels are most suitable and where not.

The relative advantage of ferries, movable bridges, swinging draw-, bascule-, lift-, and transporter-bridges, are discussed, and a table showing the general dimensions of the principal transporter-bridges, is given. The suitability of pontoon bridges is considered, and also the relative advantages of fixed bridges and tunnels, and there is a table showing the general dimensions and cost of the principal bridges over navigable waters. The paper contains a history and description of existing subaqueous highway tunnels, including the Thames, Blackwall, and Rotherhithe Tunnels, in London, the Washington Street, La Salle Street, and Van Buren Street Tunnels, in Chicago; the Glasgow Harbor Tunnel, and the Hamburg Tunnel; and a table gives their general dimensions.

Projects for subaqueous highway tunnels at New York, Boston, Chicago, Sydney, Liverpool, and Oakland, Cal., are considered. Methods of construction are discussed, which include rock tunnels,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

and, owing to their bearing on the subject, the Severn railway tunnel and the Mersey railway tunnel are described. Shield and compressed-air methods are described, as well as roof-shield, cut-and-cover, inside coffer-dam, and special methods, such as those used in the Harlem River crossing of the New York subway and the Detroit Tunnel. The caisson method used in Paris is considered, and also the freezing method.

Various types of lining—cast iron, brick, concrete, wrought steel, etc.—are described.

The shape of the section, means of access by inclines or shafts, methods of water-proofing, ventilation, and drainage, are also dealt with.

Conclusions.

The paper is a compilation of the available facts relative to subaqueous highway tunnels and other subaqueous tunnels applicable to highway purposes, and the conclusion is that, owing to the development of great harbors and the consequent need for unobstructed waterways and the increasing use of motor-driven vehicles, highway tunnels have not received sufficient consideration and are worthy of more extended discussion and attention.

Introduction.

With the growth of sea-borne commerce and its concentration at great ports, the problem of crossing harbors and their tributary navigable waters has become serious. The land traffic in the vicinity of a great harbor naturally develops with the water traffic, and inter-communication for land vehicles between the sides of a land-locked harbor is essential. The motor truck is being developed very rapidly and, if adequate facilities for inter-communication are provided, will develop still more rapidly and furnish a means of transferring goods from ocean vessels to railways, warehouses, and the like, thus relieving the small harbor craft and the railways from the burden of this transfer.

Where the harbor is not crowded, and with a low range of tide and a comparative freedom from fog and ice, a steam ferry is a fairly satisfactory means of crossing, as there are no excessive gradients of approach and the motors are idle or the horses resting while crossing. If, however, the harbor is crowded with water craft, there are two lines of traffic crossing each other's path on the same plane, or what in

railway parlance would be called a level or grade crossing, the avoidance of which would necessitate an under or over crossing, either by a tunnel or a bridge. With a great range of tide, it becomes difficult to load and unload boats, on account of the length of adjustable ramp necessary to avoid excessive gradients. With fog, ice, and bad weather, the danger of collision is increased, and the delays in loading, unloading, and operating ferries are serious, all of which add greatly to the cost of trucking.

On comparatively narrow channels, movable bridges of various sorts are used, but these do not avoid altogether the level crossing, as only the smaller craft can pass without having the draw-span opened. Many similar bridges are now being built at a level which will permit tugs and harbor craft to pass under them and thus decrease the frequency of opening. A clear space of 30 ft. from the surface of the water to the lowest member of a bridge will permit most harbor craft to pass under.

With a swinging draw, the large pivot pier required is a serious obstruction to the channel, and the maximum width of waterway, allowing for piers and fenders, is about 200 ft.* With a two-leaf bascule draw, the maximum span is about 300 ft. A lift-bridge has been built at Portland, Ore., with a span of 245 ft. and a clear headway of 135 ft. It will thus be seen that, at the best, such bridges interfere with the navigation of the waterway and fail to provide the free channels that an ocean port should have; also, the frequent opening of the draw obstructs the highway, and there is always the danger of a vessel colliding with a bridge.

Where the land traffic is light, transporter-bridges have been built. These are adapted to conditions where storms make the water very rough and cause difficulties in ferry navigation, or where the stream is obstructed at times by floating ice, or where a great fluctuation in the height of the water makes the design and construction of suitable landing ramps difficult. In general, they are erected at points where the importance of the river traffic is such that it cannot be hampered by a draw-bridge, where the traffic is so light as not to warrant the building of a fixed bridge or tunnel, or where the physical difficulties in the construction of a bridge or tunnel are very great.

* A swinging draw-bridge is to be constructed at Vancouver, B. C., with a length of 581½ ft. between end bearings; this will be the longest in existence and will have a clear width of waterway of 225 ft. (*Engineering News*, February 19th, 1914.)

Table 1 gives the general dimensions of the principal transporter-bridges.

TABLE 1.—TRANSPORTER-BRIDGES.

Location.	River.	Span, in feet.	Clear height, in feet.
Widnes and Runcorn, England..............	Mersey........	1 000	82
Newport, England..............................	Osk............	645	177
Rouen, France..................................	Seine..........	644.91	167.46
Middlesbrough, England......................	Tees...........	564.75	160.9
Marseilles, France.............................	(Harbor)......	541	164.3
Bilbao, Spain....................................	Nevrion........	524.96	147.65
Nantes, France..................................	Loire..........	462.62	165
Martrou, France................................	....	448.55	164.05
Rochefort, France..............................	Charente......	415	165
Duluth, Minn., U. S. A........................	(Harbor).......	393.75	135
Kiel, Dockyards.................................	....	387.2	145
Bizerte (near Tunis, Africa)..................	....	357.63	147.65
Brest, France....................................	(Harbor)......	357.6	144.4
Osten, Germany.................................	Oste...........	262.5	83.8

It will be seen from Table 1 that the longest span is 1 000 ft., between Widnes and Runcorn,* over the Mersey, shown by Fig. 1. The only example in America is at Duluth, with a span of 393.75 ft.†

Most of the transporter-bridges in Table 1 are over harbors, canals, arms of the sea, or inland waters, and practically no through highway for ocean-going vessels is spanned by such a structure. There are many places where no other form of construction is warranted by the traffic; but, at the best, transporter-bridges are only suitable for comparatively narrow channels, where neither the water nor land traffic is heavy, and not for the crowded conditions of a great ocean port.

Pontoon bridges, with movable sections which can be floated to one side to permit the passing of boats, have been built. Such bridges are generally used in sparsely settled regions, where the land traffic is light; they are more or less temporary in character, and are a cheap substitute for a more permanent structure where the traffic does not warrant the heavier expenditure. However, a bridge of this type, crossing the Golden Horn, at Constantinople, which is more permanent in character, has recently been completed. The length between abutments is 1 530.84 ft., and the width of roadway, 82 ft. For small boats there are two spans of 39 ft. having a clear headway of 17½ ft., and a draw section with a clear opening of 205 ft.‡

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXV, p. 87.

† *Transactions*, Am. Soc. C. E., Vol. LV, p. 322.

‡ *The Engineer*, December 6th, 1912.

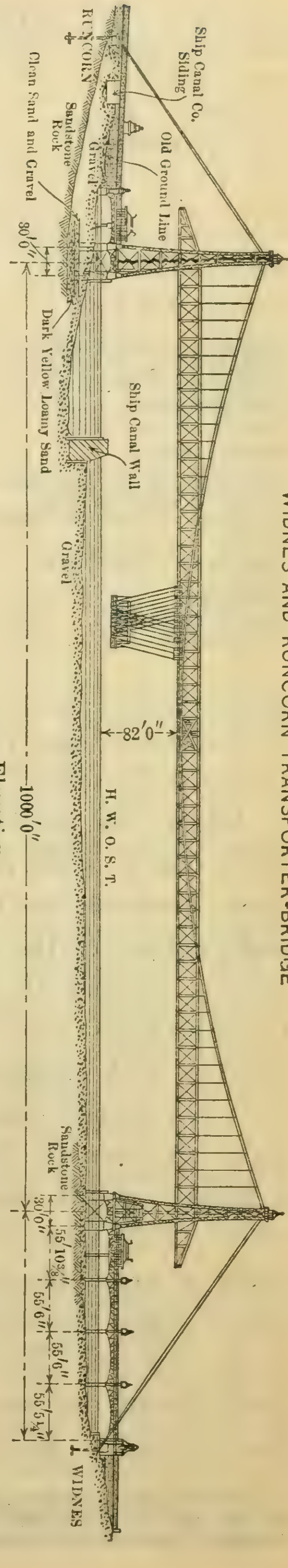
Numerous bridges of this type have been built in India.* In Calcutta one has been designed with submerged pontoons anchored at a fixed level so as to avoid the change in level of the roadway with the fluctuations of the water surface.†

Several pontoon railroad bridges of considerable length have been used in America. One, with a draw section 405 ft. long, was built at Prairie du Chien, Wis., in 1874, by the Chicago, Milwaukee and St. Paul Railway Company; this company also had four other bridges of similar type. These bridges, instead of being separate pontoons acting as piers, were really continuous floating structures.

Bridges of this type are not suited to a busy harbor frequented by numerous ocean-going vessels, and where there is a heavy land traffic.

The only remaining alternatives are fixed bridges at sufficient height to clear the shipping, or tunnels or passages at a greater depth than the draft of the vessels.

In avoiding the level crossing of these two conflicting kinds of traffic, by placing the roadway above or below the water, there is an advantage in depressing it, as the top of the structure need only be about 40 ft. below low water; whereas, if elevated, the bottom must be from 135 to 175 ft. above high water. This paper, however, is not written with the view of entering into a controversy as to the relative merits of bridges and tunnels, as no general law can



WIDNES AND RUNCORN TRANSPORTER-BRIDGE

Elevation

FIG. 1.

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXIX, p. 292.

† *The Engineer*, September 13th, 1912.

TABLE 2.—PRINCIPAL BRIDGES OF THE WORLD,

Location.	Character.	Length of longest span, in feet.	Total length of main spans, in feet.	Total length, including approaches, in feet.	Clear height above high water, in feet.
Forth	Cantilever	1 700	5 349.5	8 298.42	150
Brooklyn, New York	Suspension	1 595.5	3 455.5	7 580	135
Manhattan, New York	"	1 470	2 920	6 855	....
Williamsburg, New York	"	1 600	2 793	7 308	135
Queensboro, New York	Cantilever	1 182	3 724.5	7 449	135
Poughkeepsie	"	548	3 093.75	6 777	130
Quebec	"	1 800	2 830	3 239	150
Tower, London	Bascule	200	880	2 680	{ 29.5 / 143 }
Connecting R. R., Hell Gate	Steel arch	977.5	977.5	17 908	135
Clifton, Bristol, England	Chain suspension.	702.25	....	....	245
Washington Bridge, New York	Steel arch	510	1 660	2 375	133.5
High Bridge, New York	Stone arch	80	1 450	....	100
Menai	Suspension	570	570	1 034.71	72
Britannia	Tubular.	459.25	1 513	1 841.42	103.75
Saltash-Tamar R	Lenticular	436	910	2 240	100
Portland, Oregon	Lift	245	....	....	135
" "	"	220	....	....	144

be laid down; each problem must be decided on its merits in accordance with the conditions.*

Where the conditions are favorable for a bridge, there is no doubt that it is the most desirable means of crossing a body of water. These favorable conditions are high banks, which reduce the length and gradient of the approaches; moderate spans; a suitable stratum for the support of foundations at a reasonable depth; cheap land for approaches; and the ability to place piers so that they will not interfere with navigation. When the spans must be in excess of 1 000 ft., when proper support for foundations can only be had at a great depth, or when the land near the water is low, the conditions favor a tunnel. When bridges exceed a span of 1 000 ft., they must be given considerable width, for lateral stability, and the dead loads being so great, the bridges can be given great capacity by the addition of rapid transit and street railway tracks without adding greatly to the expense. This causes an undue concentration of traffic at the ends of such bridges and adds greatly to the difficulty and cost of the terminals. In other words, instead of building a bridge with

* "Bridges over Navigable Rivers—Some Practical Considerations," by C. E. Smith, M. Am. Soc. C. E., Bridge Engineer, Missouri Pacific Railway, *Proceedings*, American Railway Engineering Association, Vol. XIV, Part 2, p. 185.

OVER WATERS FREQUENTED BY SEAGOING VESSELS.

Width, in feet.	Use.	Dates.		Depth of water, in feet.	Cost.	Gradient of approaches.
		Work begun.	Work finished.			
30	Railway.	1883	1890	200	$15 700 000	1.43%
86	Highway, footway, electric railway, trolley.	Jan. 3, 1870	May 24, 1883	62	22 400 000	3.25%
120	"	Oct. 1, 1901	Dec. 31, 1909	67	26 000 000	3.25%
118	"	Nov. 7, 1896	Dec. 19, 1903	50	23 100 000	3.00%
89.5	"	June, 1901	Mar. 30, 1909	88	17 900 000	3.50%
35	Railway.	Sept., 1886	1889	60	....	....
88	"	....	....	200	(Disaster, Aug. 29th, 1907.)	
50	Highway.	June, 1886	June, 1894	33.5	....	....
93	Railway.	July, 1912	....	108	....	1.20%
31	Highway.	1861	Dec., 1864	31	$360 000	....
80	"	July, 1886	Feb., 1889	14.5	2 851 684	....
21	Aqueduct.	....	1842	14.5	....	....
29.5	Highway.	1819	1826	30	....	Level.
2–14.5	Railway.	Apr. 13, 1846	1850	75	2 931 000	"
16.83	"	....	May, 1859	60	....	....
....	Highway.	....	1911	....	....	....
....	Highway and railway.	....	....	....	....	....

capacity to suit the needs of the traffic, or several bridges located so as to suit the convenience of the public, economical considerations necessitate the construction of a single large bridge, either of greater capacity than required, or which concentrates the traffic along a single path that would be better served by several.

One other point should be given due consideration, and that is the relative ease of destruction of a bridge, as compared with a tunnel. The severing of a single member would cause the collapse of a whole span. Although injury at a single spot might cause the flooding of a tunnel, the damage would be local and could be readily repaired. This is mentioned in view of the fact that national wars have not altogether ceased, and that those who disapprove of existing conditions sometimes express their dissatisfaction by the use of explosives.

Table 2 shows the general dimensions of some of the world's principal bridges crossing waters frequented by seagoing vessels. This table does not purport to be exhaustive, but merely gives typical examples of the most important bridges of the various classes, some being included merely on account of their historical significance. The physical conditions are such at certain of the bridges in Table 2

that a tunnel could not readily be substituted; at the Forth Bridge, on account of the great depth of water, which is more than 200 ft.; at the connecting railroad bridge at Hell Gate, for the same reason; at the Poughkeepsie Bridge, Washington Bridge, High Bridge, etc., on account of the height of the river banks. On the other hand, at many of the other bridge locations, in view of the modern state of the art, tunnels could be constructed more advantageously than bridges.*

Highway Tunnels: History.

London.—It is a curious fact that one of the first subaqueous tunnels, and the first on which a shield was used, was constructed as a highway tunnel, but was never used for that purpose. This was the first Thames Tunnel, built by the inventor of the shield, the late Sir Marc Isambard Brunel. His patent for a shield was No. 4 204, of 1818. Owing to London Bridge marking the head of navigation for seagoing vessels, the development of the docks and tributary industries was necessarily below this point, so that the need of some means of crossing the river below London Bridge is evident. This is indicated in Fig. 2, which shows the position of the Tower Bridge and the various tunnels below London Bridge. The Thames Tunnel, Fig. 3, crosses the river between Rotherhithe and Wapping, about 1½ miles below London Bridge. Work was started in 1825 and, after being suspended several times owing to physical and financial difficulties, was completed in 1843. In section it is one of the widest openings ever constructed, the total width of excavation being 37 ft. 6 in., and in height 22 ft. 3 in., separated by a wall 4 ft. thick, pierced by 64 arched openings, each of 4 ft. span. The length is 1 200 ft. It was proposed to have spiral approaches, but this was never done; the construction shafts, however, were fitted with stairways. In 1866, the tunnel was sold to the East London Railway Company, and since that time this company has operated its trains through it. Its cost, including shafts, was £1 300 per lin. yd.

As the Thames Tunnel had been diverted from its intended use, the vehicular traffic still had no facilities below London Bridge, but its needs were partly met by the Tower Bridge, about ½ mile below

* "Bridges and Ferry Bridges; Tunnels under Waterways used for Ocean Navigation—Economic and Technical Study," a report by Baurat Wendemuth, International Congress of Navigation, 1912.

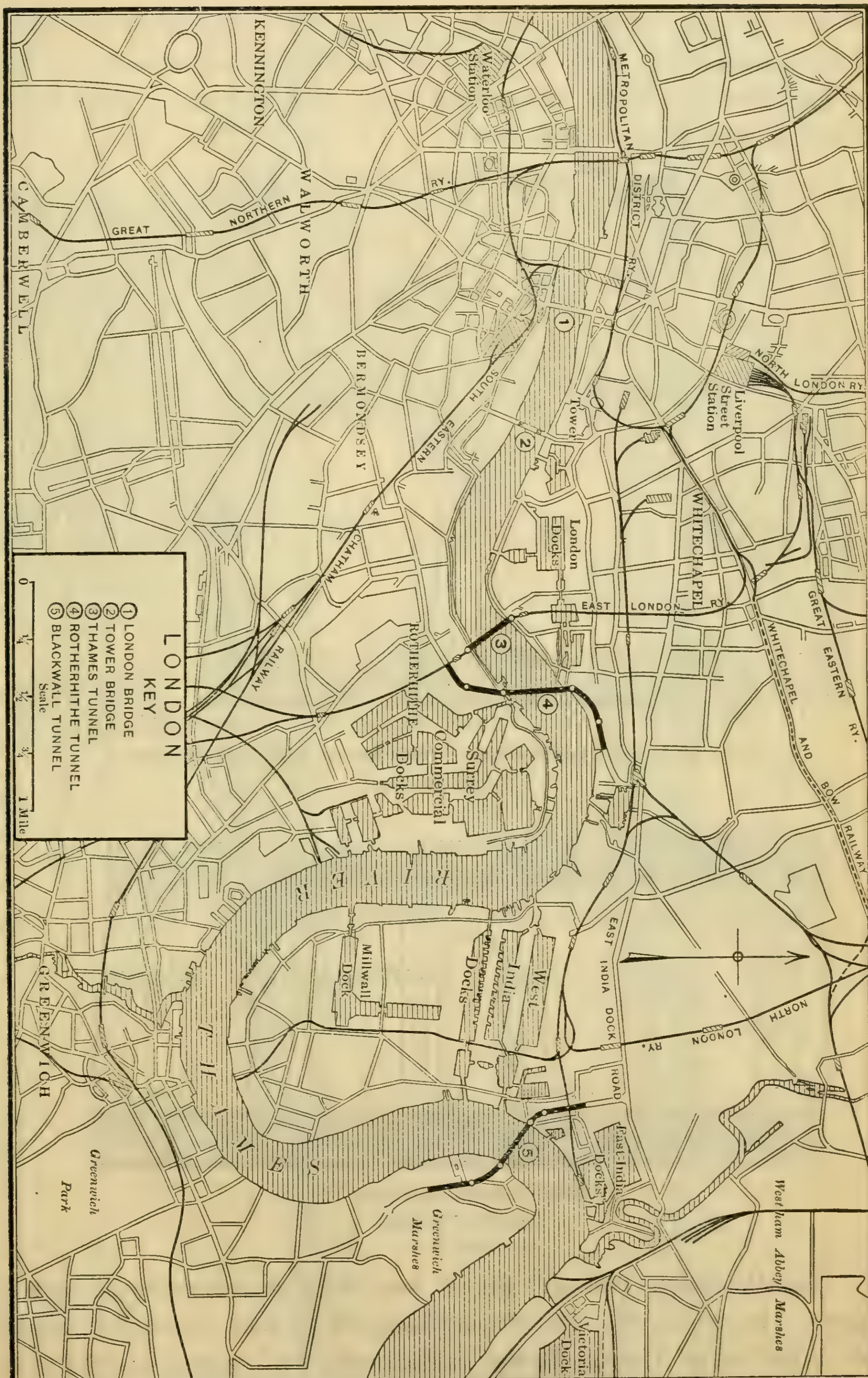
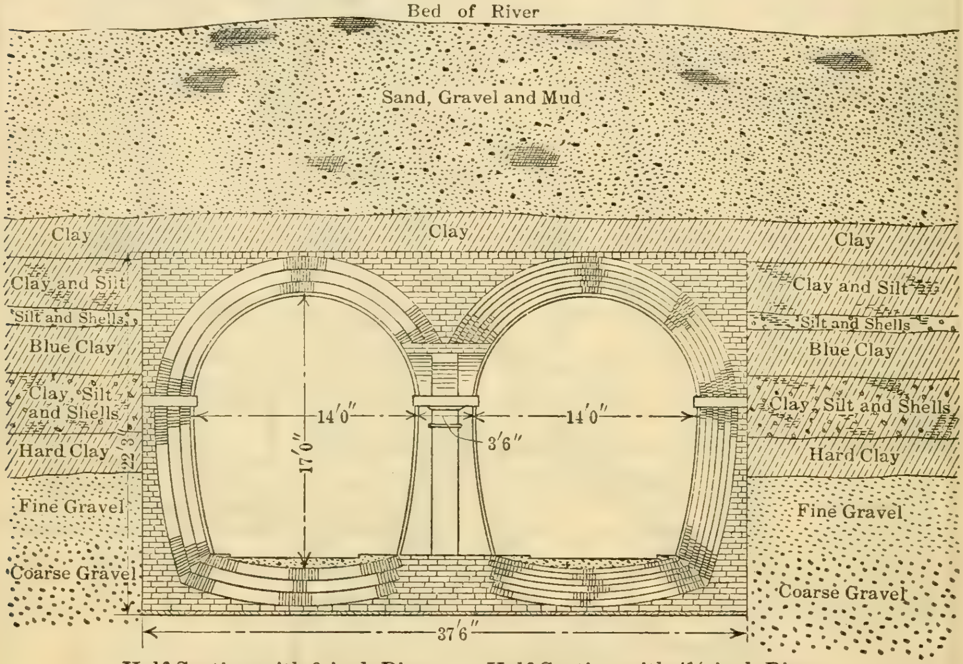

LONDON
KEY
1 LONDON BRIDGE
2 TOWER BRIDGE
3 THAMES TUNNEL
4 ROTHERHITHE TUNNEL
5 BLACKWALL TUNNEL
0 1/4 1/2 3/4 1 Mile
Scale
KENNINGTON
CAMBERWELL
WALWORTH
BERMONDSEY
ROTHERHITHE
GREENWICH
WHITECHAPEL
Waterloo Station
Liverpool Street Station
Tower
London Docks
Surrey Commercial Docks
West India Docks
Millwall Dock
East India Docks
Victoria Dock
Greenwich Park
Greenwich Marshes
West ham Abbey Marshes
RIVER THAMES
GREAT NORTHERN RY.
METROPOLITAN DISTRICT RY.
NORTH LONDON RY.
EAST LONDON RY.
SOUTH EASTERN
CHATHAM RAILWAY
GREAT EASTERN RY.
WHITECHAPEL AND BOW RAILWAY
EAST INDIA DOCK ROAD
NORTH LONDON RY.

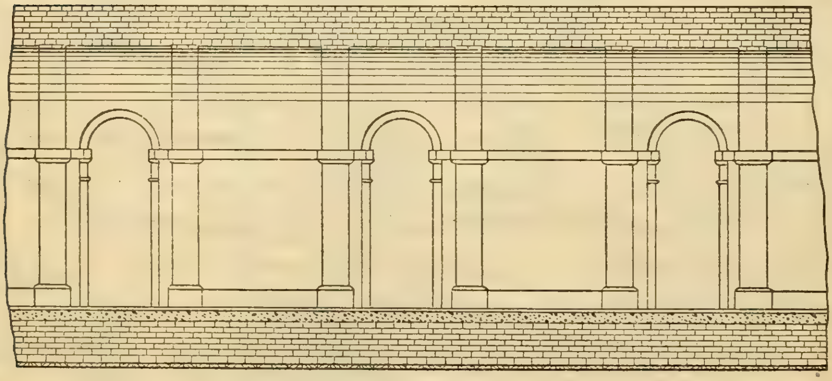
FIG. 2.

London Bridge, opened to traffic in 1894. This consists of a double-leaf bascule draw, with a clear span between towers of 200 ft., and a high-level footbridge connecting the towers, leaving a clear head-room of 139½ ft. above high water.



Half Section with 9-inch Rings Half Section with 4½-inch Rings

CROSS-SECTION



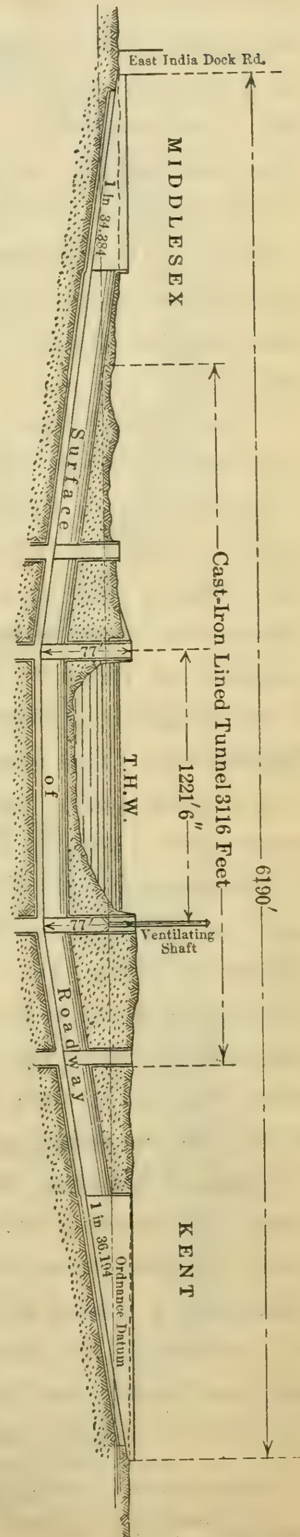
LONGITUDINAL SECTION

THAMES TUNNEL

FIG. 3.

The Blackwall Tunnel,* Fig. 4, was the next project to be undertaken. It crosses the river from Blackwall to East Greenwich, about 6¼ miles below London Bridge. The work was started in 1891 and

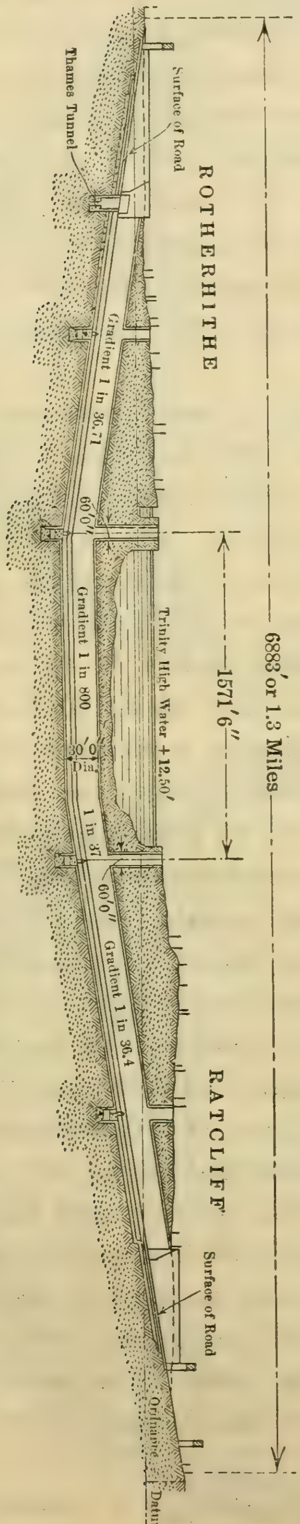
* *Minutes of Proceedings*, Inst. C. E., Vol. CXXX, p. 50.



PROFILE, BLACKWALL TUNNEL

Scales: Horizontal 0 100 500 1000 1200 Ft.; Vertical 0 50 100 150 200 240 Ft.

FIG. 4.



PROFILE, ROTHERHITHE TUNNEL

Scales: Horizontal 0 100 500 1000 1200 Ft.; Vertical 0 50 100 150 200 240 Ft.

FIG. 5.

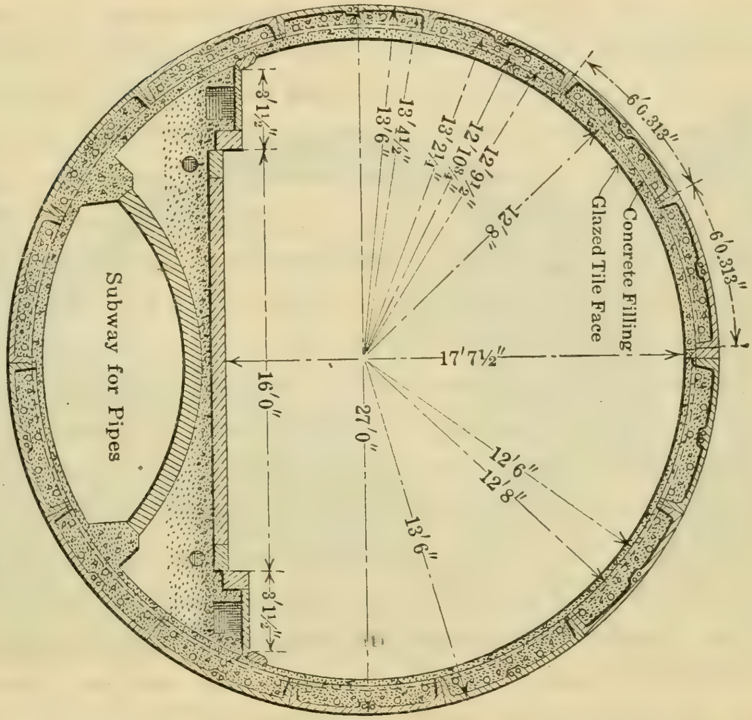
the tunnel was opened to traffic in 1897. It consists of a shield-driven, iron-lined tunnel (Fig. 6), 27 ft. outside diameter, with an internal diameter of 24 ft. 3 in. It is lined with concrete faced with glazed tile, has a roadway 16 ft. wide, and two foot-paths, each 3 ft. 1½ in. wide. The total length, including approaches, is 6 200 ft.; the length from portal to portal is 4 465 ft.; and the length under the waterway is 1 220 ft. The maximum gradient of approach is 1 in 36. Changes in direction were not made by driving the tunnel in curves, but by angles at the shafts. The cost was £871 000, amounting to about $685 per lin. ft.

The Rotherhithe Tunnel,* Fig. 5, was commenced in 1904 and completed in 1908. It crosses the river diagonally from Rotherhithe to Ratcliff, about 2¼ miles below London Bridge. The work was very similar to that at the Blackwall Tunnel. It is a shield-driven, iron-lined tunnel, with an external diameter of 30 ft. and an inside diameter (inside the concrete and tile lining) of 27 ft. It is provided with a roadway 16 ft. wide, and two walks, each 4 ft. 8½ in. wide. Figs. 8 and 9 show the Stepney Approach, and the spiral stairs at Shaft 2, Rotherhithe. The total length, including approaches, is 6 883 ft.; the length from portal to portal is 4 930 ft.; and the length under the waterway is 1 480 ft. The cost was £1 088 484, or about $930 per lin. ft.

Chicago.—In the United States the first subaqueous highway tunnel was the Washington Street Tunnel, crossing the Chicago River, in Chicago.† (Fig. 10.) This is not properly a tunnel at all, but a subaqueous passage built in an open trench between coffer-dams. It was built by the City of Chicago in 1866 to 1869, and had two roadways, each 11 ft. wide and 13 ft. high, and a footway 10 ft. wide and 10 ft. high, as shown by Fig. 11. It was at first used quite extensively for general highway traffic, and, after the fire in 1871, was the only means of communication between the west side and the business district, for some time, until the burned bridges could be restored. Incidentally, a tunnel is less likely to be damaged by fire or flood than is a bridge. This tunnel, later, fell into a bad state of repair and was used much less frequently, the traffic being largely diverted to the nearby bridges. At the time of the change of power from horse

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXV, p. 190.

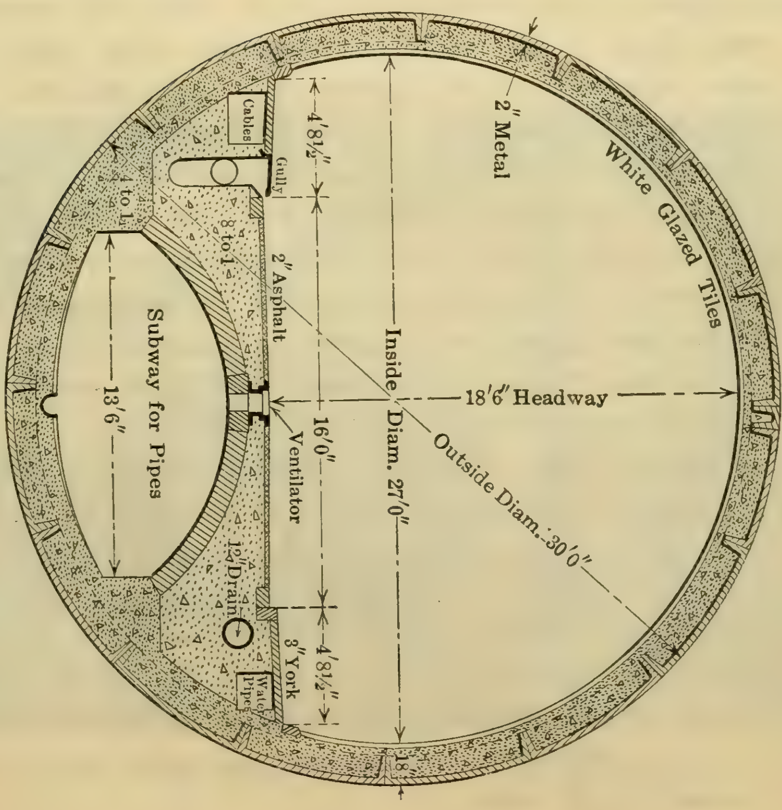
† "Chicago River Tunnels: Their History and Method of Construction," by William Artingstall, *Journal*, West. Soc. of Engrs., Vol. XVI, p. 869.



1½-Inch Castings 2-Inch Castings

CROSS-SECTION, BLACKWALL TUNNEL

FIG. 6.



CROSS-SECTION, ROTHERHITHE TUNNEL

FIG. 7.

to cable on the Chicago street railways, permission was obtained to utilize the tunnel, one of the conditions being that the roof be lowered so as to give a depth of 17 ft. of water in the river, instead of 14 ft. This use of the tunnel was desirable, as the difficulties of operating a cable railway over a movable bridge were great. Since the substitution of electric traction for the cables, the tunnels are not so necessary, but the advantages of uninterrupted service still remain. The tunnel roof was lowered in 1889, the cost being about $135 000. This work was done by enclosing the tunnel between coffer-dams—only one-half of the river being obstructed at one time. The masonry arched roof was removed and replaced with 20-in. steel I-beams, 36 in. apart, with jack-arches between. At the same time, the dividing wall between the two roadways was removed and the tunnel was given a single opening of 20 ft. clear span. The invert was also cut out and lowered 2 ft. 6 in. in order to provide the necessary headroom. The use of the tunnel by vehicles and pedestrians was discontinued; a draw-bridge was constructed nearby; and the footway was used as a pipe gallery. In 1906 the tunnel was again reconstructed, in order to provide a navigable depth of 26 ft. instead of 17 ft. This was accomplished by cutting chases in the walls and inserting 33-in. plate girders, 48 in. apart, supporting a new masonry roof, after which the old roof was removed by breaking it up, without blasting, and dredging from the exterior. The side-walls were then underpinned and a new invert was constructed, as shown by Fig. 12. In 1909 further alterations were made in order to enable the tunnel eventually to form part of a subway system in the business district, and this necessitated building temporary inclined approaches to enable the surface cars to continue the use of the tunnel, pending the construction of the subway system. The cost of this work was about $675 000.

La Salle Street Tunnel (*Chicago*).—The construction of the original La Salle Street Tunnel was commenced by the City of Chicago in 1869, and it was opened for traffic on July 4th, 1871. The type of construction was similar to that of the Washington Street Tunnel. The total length is 1 887 ft., and the gradients of approach are 1 in 20. In section, it consisted of two roadways, each 11 ft. wide, and a footway 11 ft. wide and 10 ft. high, as shown by Fig. 13. The original cost was $566 000. The tunnel was given over to cable-operated street cars in 1889.

FIG. 8.—ROTHERHITHE TUNNEL: OPEN APPROACH, STEPNEY.

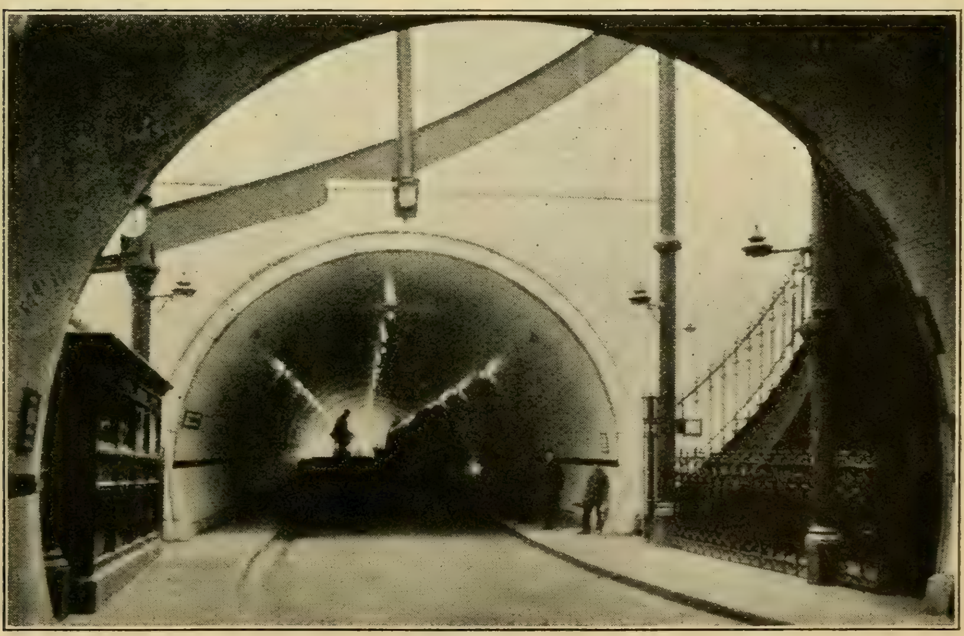


FIG. 9.—ROTHERHITHE TUNNEL: SPIRAL STAIRS, SHAFT 2.

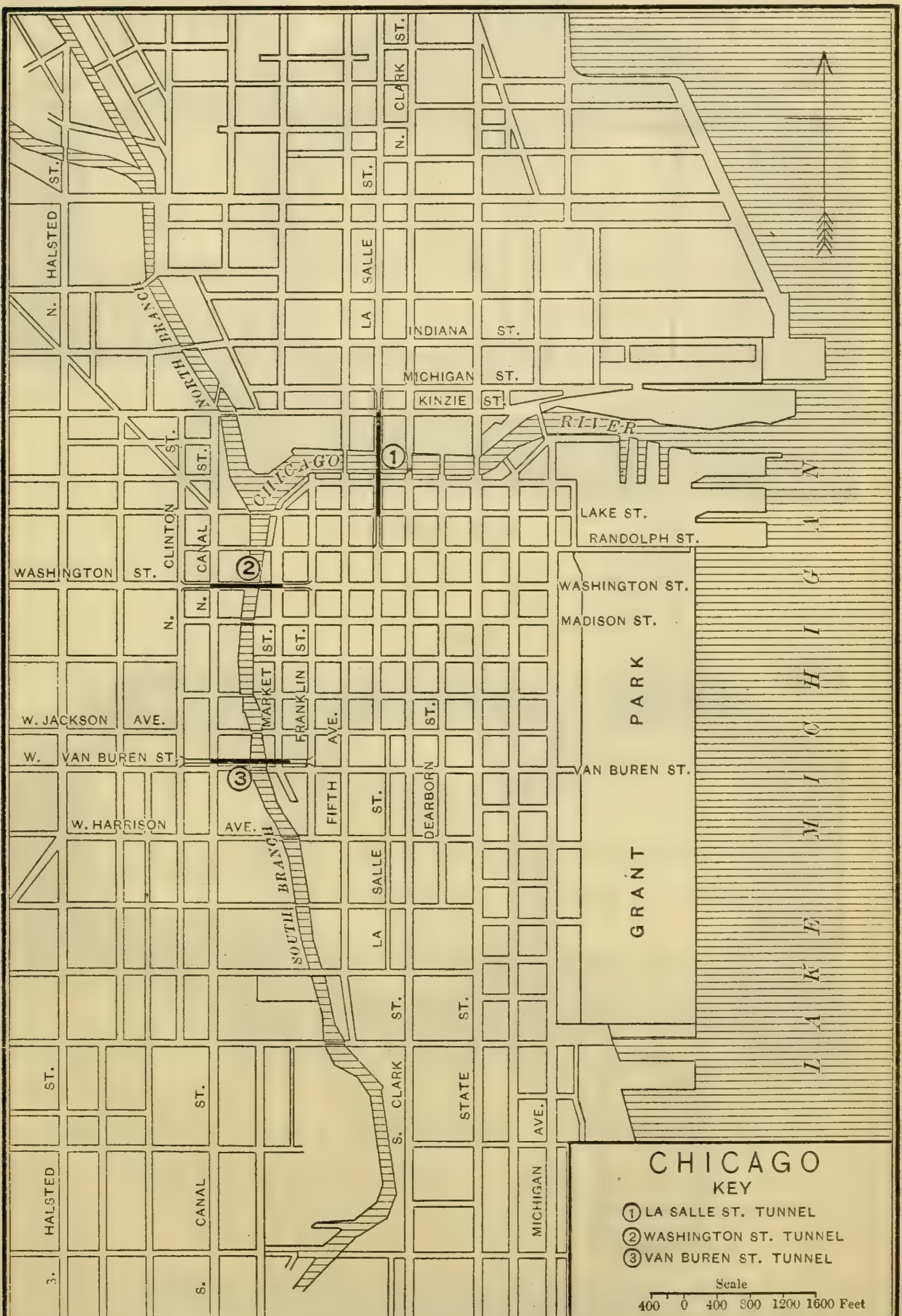
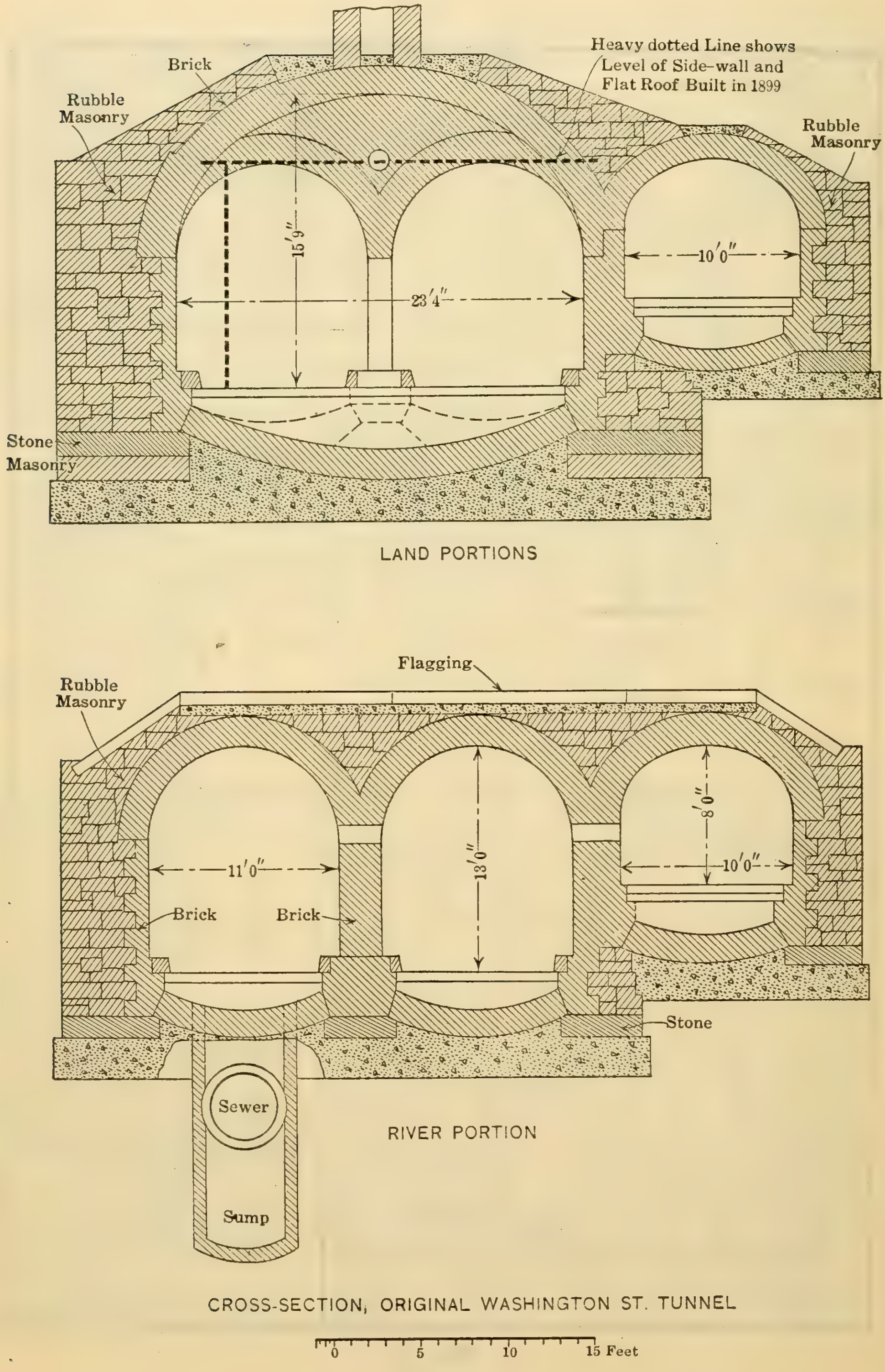

CHICAGO
KEY
1 LA SALLE ST. TUNNEL
2 WASHINGTON ST. TUNNEL
3 VAN BUREN ST. TUNNEL
Scale
400 0 400 800 1200 1600 Feet
CHICAGO RIVER
NORTH BRANCH
SOUTH BRANCH
LAKE MICHIGAN
GRANT PARK
N. HALSTED ST.
N. CLARK ST.
LA SALLE ST.
INDIANA ST.
MICHIGAN ST.
KINZIE ST.
LAKE ST.
RANDOLPH ST.
WASHINGTON ST.
MADISON ST.
VAN BUREN ST.
N. CLINTON ST.
N. CANAL ST.
MARKET ST.
FRANKLIN ST.
FIFTH AVE.
DEARBORN ST.
W. JACKSON AVE.
W. VAN BUREN ST.
W. HARRISON AVE.
LA SALLE ST.
S. CLARK ST.
STATE ST.
MICHIGAN AVE.
S. HALSTED ST.
S. CANAL ST.

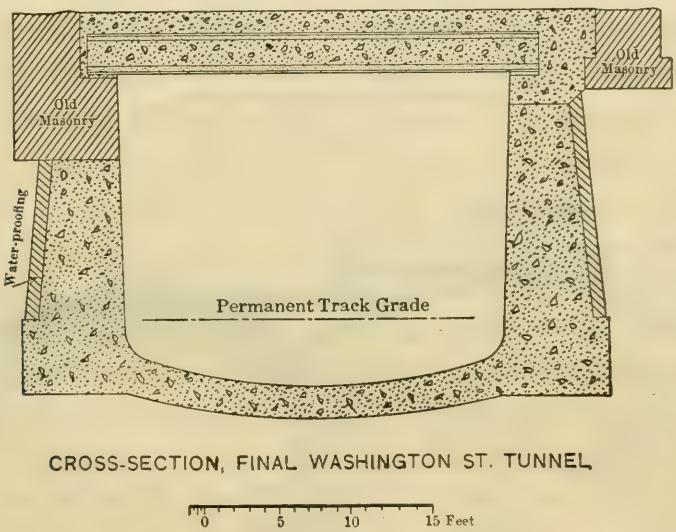
Fig. 10.



CROSS-SECTION, ORIGINAL WASHINGTON ST. TUNNEL

FIG. 11.

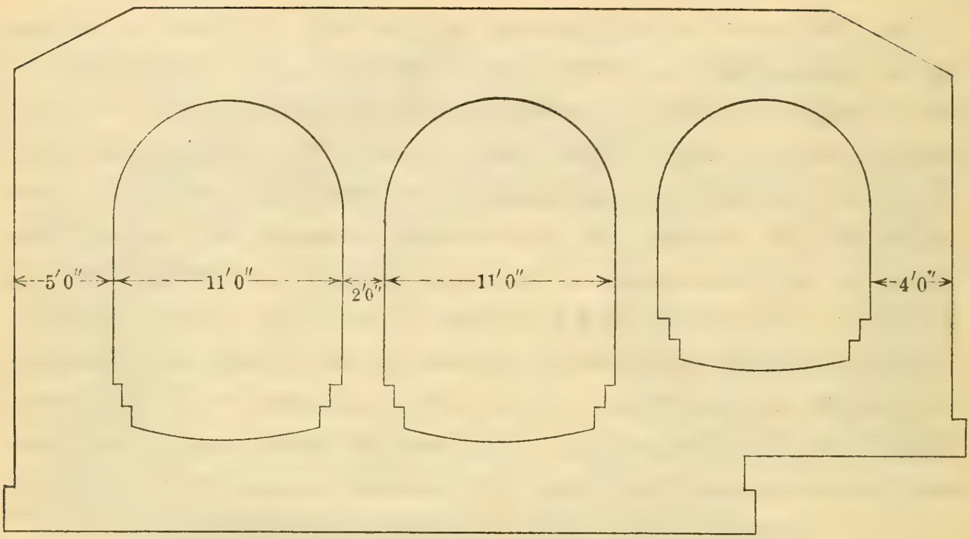
In 1906 work on the lowering of the roof, in order to increase the navigable depth of water from 18 to 26 ft., was commenced. It was intended to follow a method similar to that used for the Washington Street Tunnel. Inside the tunnel, 30-in. steel girders were to rest on the old west wall, and a new brick wall was to be placed inside the old footway, the latter being retained as a pipe gallery. Owing to the development of excessive leaks in executing this work, the tunnel was flooded and a change of plan was made. A method of doing the work by enclosing sections in coffer-dams was considered, but, as the United States Government engineers would not permit coffer-dams for a distance of 60 ft. from the dock lines, it was necessary to devise some other plan. The method adopted was to construct



CROSS-SECTION, FINAL WASHINGTON ST. TUNNEL

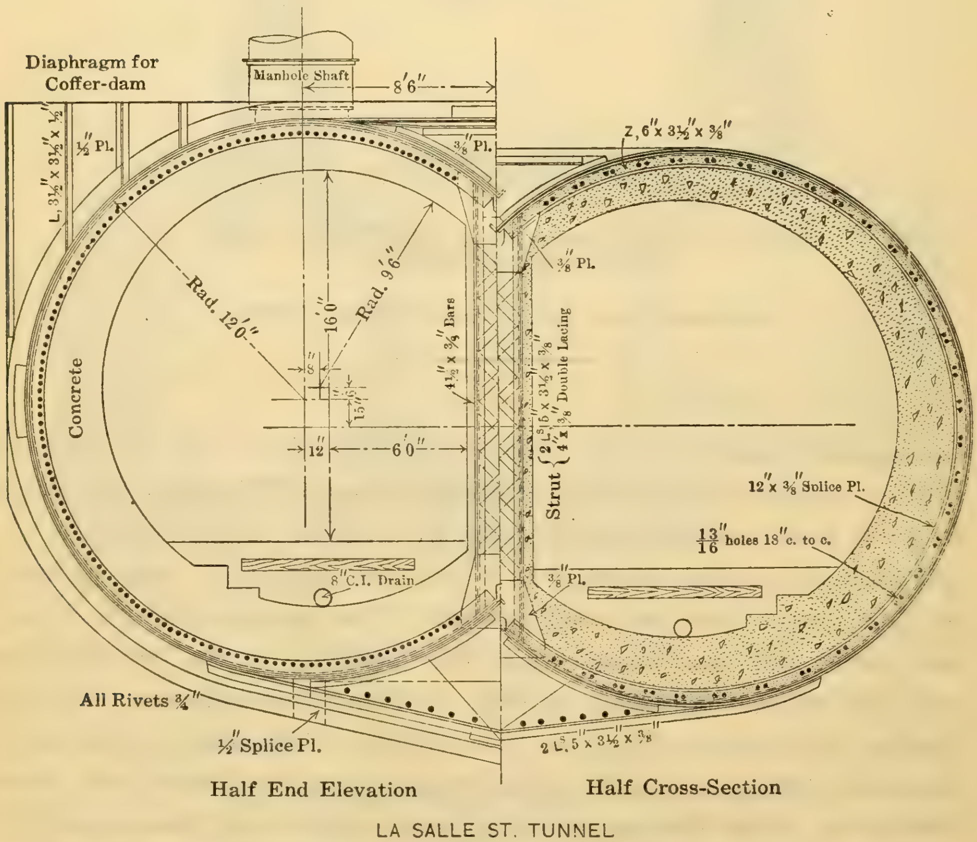
FIG. 12.

two parallel steel cylinders, each comprising about 75% of a circle, with a common longitudinal stiffening truss. This shell was built at a dry dock, and was 278 ft. long, 27 ft. high, and 41 ft. wide. It was made up of ⅜-in. plates, 7 ft. 6 in. wide, and 19 ft. long, stiffened by Z-bars and plate and angle gussets. The concrete for the invert and center wall was placed while the shell was in the dry dock, as were the timber bulkheads in each end, and the temporary internal bracing, as shown by Fig. 14. The weight of the steel in the tubes was about 500 tons, and the total weight when first floated was about 3 000 tons. After floating, the shell was towed to near the site of the tunnel. Internal concrete was then added until the tops of the tubes barely projected above the water, the total displacement being 7 852



CROSS-SECTION, ORIGINAL LA SALLE ST. TUNNEL

FIG. 13.



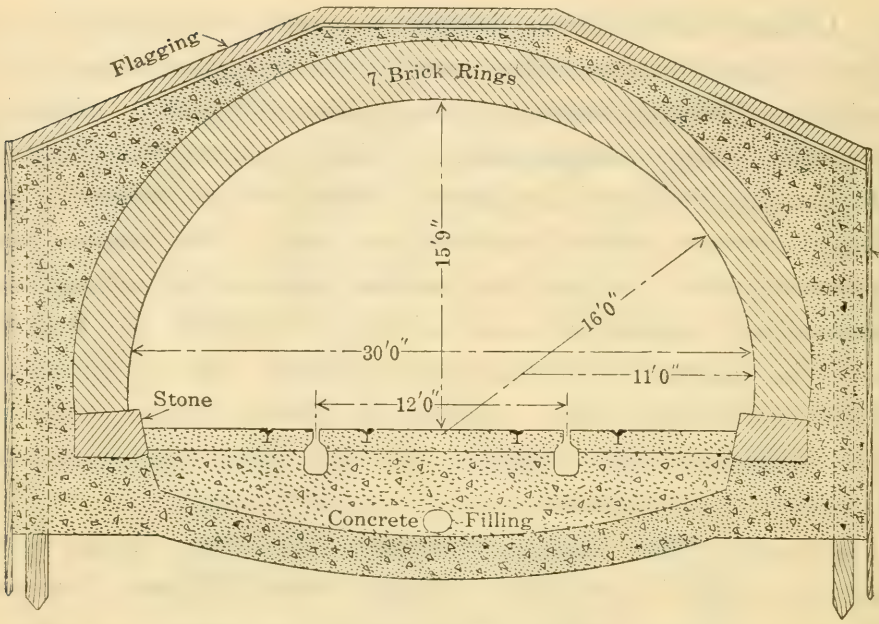
LA SALLE ST. TUNNEL

FIG. 14.

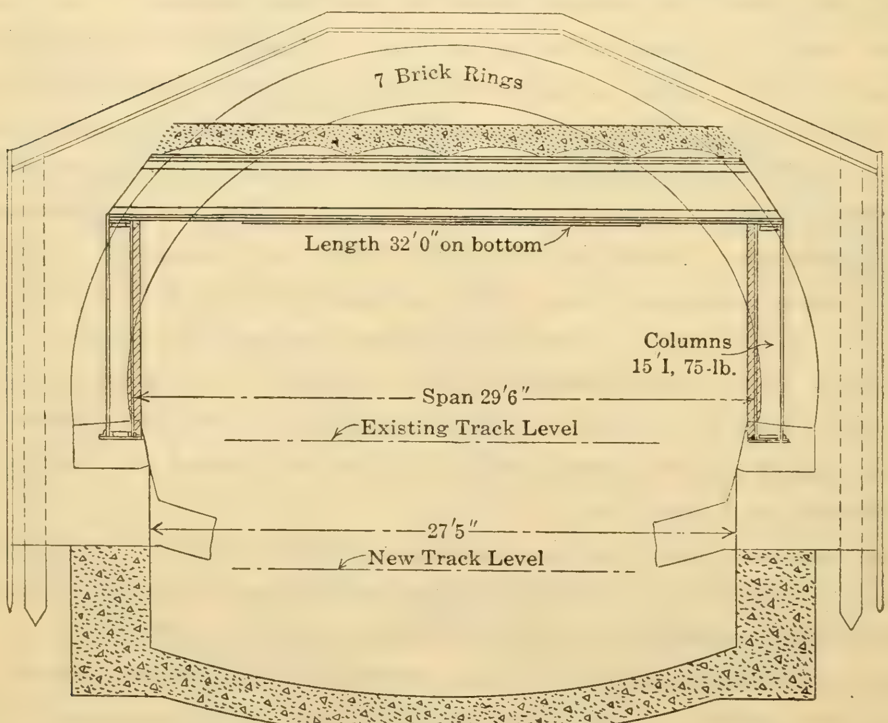
tons. A trench having been dredged to grade, the current in the river was stopped by closing the gates of the Chicago drainage canal at Lockport. The shell was towed into position and sunk by admitting water into interior compartments. The structure was held in position by guy lines, and the rate of sinking was controlled by suspending it between pairs of barges at each end. After sinking, sand was pumped under it to give it a uniform bearing; the remainder of the back-filling consisted of sand and clay. The shell was then unwatered, the internal concreting was completed, and the connection with the land section made. Unlike the Detroit Tunnel, the shell was not surrounded with concrete, the only exterior concrete being that required to form the base and to level off the top.

Van Buren Street Tunnel (Chicago).—The Van Buren Street Tunnel was built by the West Chicago Street Railway Company. Work was started in 1889, and the tunnel was opened to traffic on April 23d, 1894. It differed from the previous tunnels in that both tracks were in a single opening having a three-centered brick-arched section of 30 ft. span, as shown on Fig. 15. It contained two tracks for cable cars. It was constructed by the coffer-dam method, and was 1 517 ft. long, including approaches. This tunnel was remodeled in 1906 to provide 26 ft. of water in the river. This was accomplished by constructing, inside the old arch, a new roof composed of 32-in. steel girders resting on 15-in., 80-lb. I-beam columns, 4 ft. 3 in. from center to center, with concrete jack-arches between the girders. The side-walls were then underpinned and extended to a greater depth, and the invert was rebuilt at the lower level. After the completion of the new roof, the upper portions of the old tunnel were removed by blasting and dredging.

General.—At the time the highway tunnels in Chicago were built, the art of constructing movable bridges had not been largely developed, and electric propulsion for street cars was unknown. With the development of cable traction, the cable companies obtained permission to use these tunnels, as such lines could not very well be operated over movable bridges. At the same time, the increasing number of bridges, together with improvements in the mechanism for operating them—which reduced the delays due to the draw being open—and the avoidance of the heavy gradients of the tunnel approaches. led to their



CROSS-SECTION, ORIGINAL VAN BUREN ST. TUNNEL



CROSS-SECTION, FINAL VAN BUREN ST. TUNNEL

FIG. 15.

diminishing use by vehicular traffic, and finally to their being turned over exclusively to the street-car lines.

The original Washington Street Tunnel was in very leaky condition from the beginning, and in the winter great trouble was caused by the formation of ice—so much so, that in severe weather the accumulation of ice and icicles could not be removed fast enough, and the tunnel had to be closed to traffic.

Glasgow.—In Glasgow, a tunnel for vehicles was constructed across the Clyde in 1890-93. Access to this tunnel, instead of being obtained by inclined approaches, is by elevators in shafts. The tunnel consists of three separate cast-iron tubes, each having an internal diameter of 16 ft., the center tube being used as a footway and the outer ones for vehicles, each in a single direction. Fig. 16 shows the profile. The footway tube has inclined approaches and stairs. The shafts are circular, 76 ft. inside diameter, and 80 ft. outside diameter. The river is 415 ft. wide, and the length of the tunnel from shaft to shaft is 700 ft. This tunnel was constructed by private capital, but, owing to competition by a municipal ferry, its use has been discontinued. It is now, however, proposed to re-open it, the City Corporation having arranged with the Glasgow Harbor Tunnel Company to operate it for a period of one year after the date of re-opening, on condition that, on the expiration of that time, the Corporation shall have the option of purchasing the whole undertaking for £100 000, although the original cost was £287 000.*

Hamburg.—A highway tunnel, Fig. 17, very similar to that at Glasgow, has just been completed under the Elbe at Hamburg.† Access to this tunnel is also obtained by elevators in shafts, and the total length from center to center of shafts is 1 471 ft. Each shaft (72 ft. inside diameter and 84 ft. outside), contains four elevators for vehicles and two for foot passengers. The lift of the elevators is 78 ft. The lining is composed of structural-steel plates and angles, the outer diameter being 19.7 ft. It has a lining of concrete and tile, and the roadway is only wide enough for a single vehicle, there being a separate tube for traffic in each direction. Fig. 18 is a cross-section. The general dimensions of the subaqueous highway tunnels of the world are given in Table 3.

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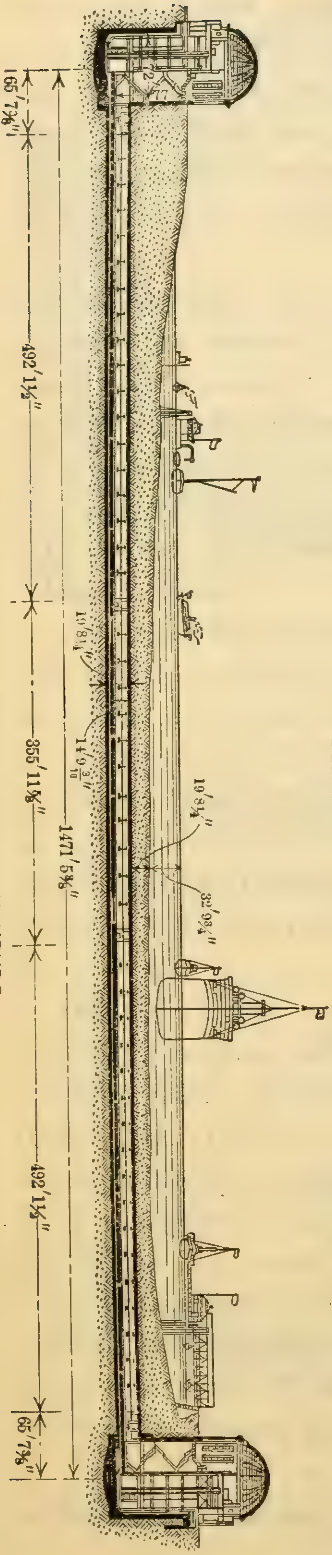
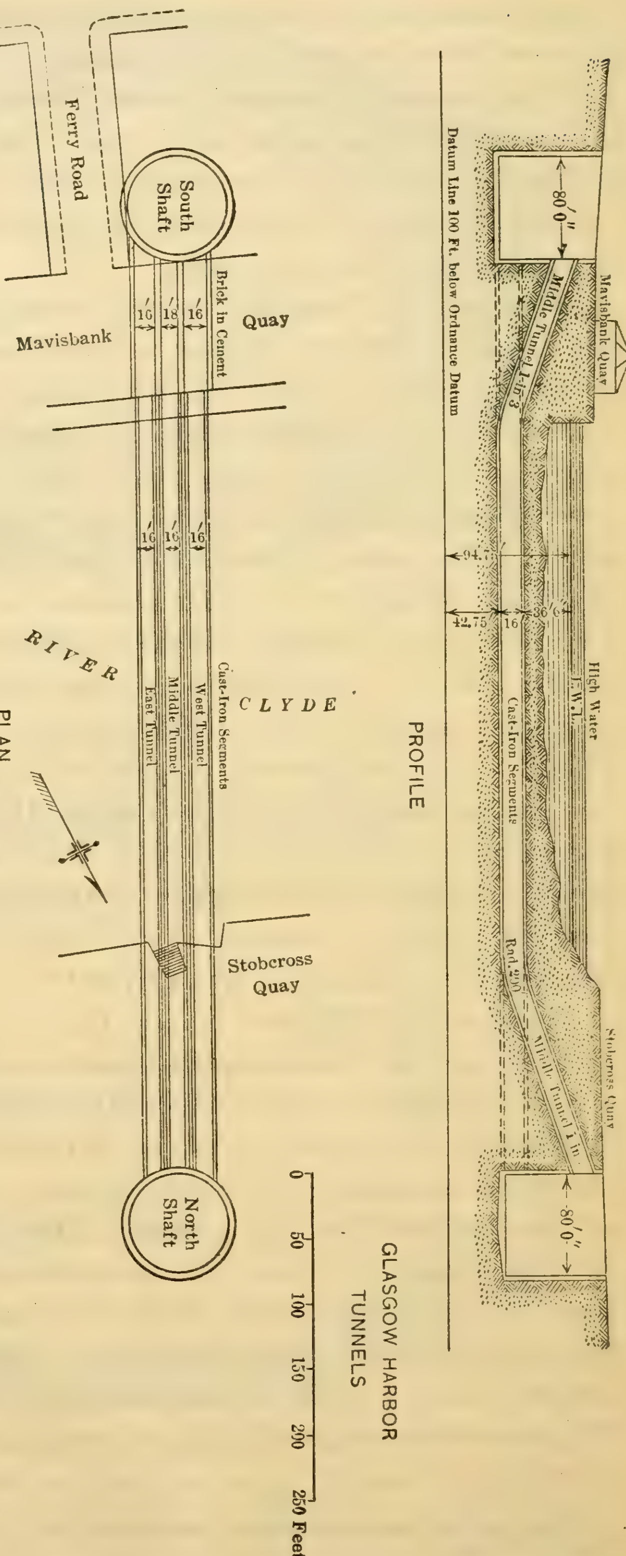
* *Engineering*, May 10th and 31st, June 14th and 28th, 1895, contains a description of the shafts, lifts, etc.

† *Zeitschrift*, Verein Deutscher Ingenieure, August 17th, 1912.



PROFILE OF ELBE TUNNEL AT HAMBURG

FIG. 17.



PLAN

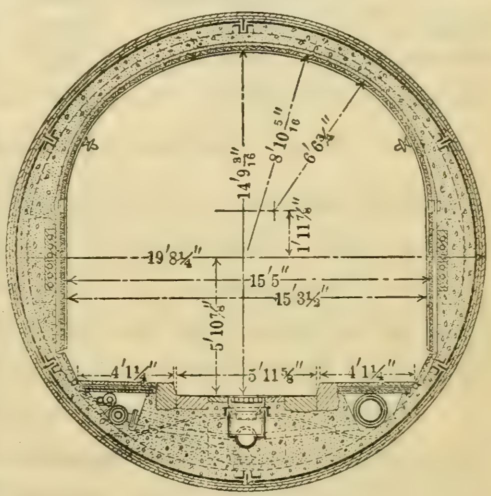
FIG. 16.

Projects.

From the preceding it will be seen that highway tunnels have not had a very fortunate history. The Thames Tunnel, the first ever built, was never utilized for the purpose intended, but was first used by a steam railway, and now by a railway operated by electric power. The tunnel, however, is now performing a useful function, and is practically in as good condition as when completed, 70 years ago, carrying more vehicles, each of much greater weight, than was contemplated originally, and it is not probable that as much could be said of a bridge, other than one of masonry.

The Chicago tunnels have had to be rebuilt to provide greater depth of water (one of them being rebuilt twice), and their use for general highway traffic has been discontinued. They are now being utilized only by the street cars.

The use of the Glasgow Tunnel has been discontinued, owing to municipal competition, and only the Blackwall, Rotherhithe, and Hamburg Tunnels remain in successful operation. Nevertheless, various projects are under consideration for the construction of highway tunnels, and therefore the subject is worthy of consideration.



CROSS-SECTION, OF ELBE TUNNEL AT HAMBURG

Fig. 18.

New York.—Some means other than ferries for vehicles to cross the Hudson (North) River at New York has long been needed. As the river forms the boundary between the States of New York and New Jersey, such means of crossing must be provided by the joint action of the two States or by a private corporation. This subject is now under investigation by the two States, through the medium of the New Jersey Interstate Bridge and Tunnel Commission and the New York Interstate Bridge and Tunnel Commission. These two Commissions have agreed that, if a bridge is to be built, it should be as far south as physical conditions will permit. As no suitable foundation can be obtained at points where piers would be permitted in the lower

TABLE 3.—GENERAL DIMENSIONS OF THE

Location.	Method of construction.	Dates.		Length under water-way, in feet.	Length, portal to portal, in feet.	Total length including approaches, in feet.	Depth to invert, in feet.
		Work begun.	Work finished.				
Thames	Shield	1825	1843	925	1 200	......	65
Blackwall	Shield	1891	1897	1 220	4 465	6 200	80
Rotherhithe	Shield	1904	1908	1 480	4 930	6 883	75
Chicago, Washington St.	Coffer-dam	1866	1869	152	934	1 608	36
Chicago, Washington St.	Reconstructed	1906	1910	152	......	1 550	53
Chicago, La Salle St.	Coffer-dam	1869	1871	285	1 505	1 890	38
" " "	Reconstructed	1906	....	285	.....	2 030	50
" Van Buren St.	Coffer-dam	1889	1894	184	920	1 514	44
Glasgow	Shield	1890	1893	415	700	......	65
Hamburg	Shield	1907	1911	1 214	1 471	......	72
Severn	Rock	1873	July 1, 1887	2¼ miles, 1 650 ft.	22 998	6½ miles	156

reaches of the river, a location at about Fifty-eighth Street has been recommended. This will require a span of 2 880 ft.

These Commissions have recommended a location for a tunnel crossing the river from the foot of Canal Street, New York, to about the foot of Thirteenth Street, Jersey City, as shown on the map, Fig. 19. This location is the most suitable, as it is in the average path of the present lines of communication for vehicles crossing the river, and is within easy reach of the present bridges over the East River. The New York approach to such a tunnel will come to the surface at about the intersection of Varick and Canal Streets. Varick Street is now being widened, and will connect with the extension of Seventh Avenue, which is now under way.

The present traffic crossing the river at New York is about 20 000 vehicles daily.

Boston.—In Boston there has been some agitation for a highway tunnel from Boston to East Boston. Gen. J. G. Foster, when United States Engineer Officer at that port, in 1868, prepared a design for such a tunnel.

Chicago.—In Chicago it has been proposed to build a tunnel under the Chicago River in order to connect the Boulevard and Park System. Two tunnels, each 31 ft. in diameter, have been suggested.

WORLD'S SUBAQUEOUS HIGHWAY TUNNELS.

Depth of water, in feet.	No. of tunnels.	Size of Tunnels.		Kind of lining.	Rate of gradient of approach.	Character of strata tunneled through.	Cost.
		Internal.	External.				
32	Twin	16′ 4″ × 13′ 9″	22′ 3″ × 37′ 6″	Brick	......	Clay & gravel	$2 110 per ft.
40	1	24′ 3″ dia.	27′ 0″ dia.	Cast iron	1:36	Cl., sand & gr.	4 242 000
40	1	27′ 0″ dia.	30′ 0″ dia.	Cast iron	1:36.5	" " " "	5 300 000
14	3	2–13′ 0″ × 11′ 0″ 1–10′ 0″ × 10′ 0″	....	Brick	1:16	Stiff blue clay	512 707
26	1	1–21′ 0″ × 25′ 0″	....	Concrete	1:10	" " "	....
17	3	....	....	....	1:20	" " "	566 000
27	2	19′ 0″	24′ 0″	Concrete	1:33.3	" " "	....
18	1	15′ 9″ × 30′ 0″	28′ 0″ × 42′ 0″	Brick & con.	1:10	Soft clay	1 800 000
49	3	16′ 0″ dia.	17′ 0″ dia.	Cast iron	......	Sand & gr.	1 398 000
33	2	14.8′	19.7′	Structural steel.	....	Sand	2 386 000
96.30	1	25′ 0″ × 26′ 0″	30′ 0″ × 31′ 0″	Brick	1:90	Sandstone shale	7 300 000

Sydney.—At Sydney, New South Wales, a highway tunnel has been proposed to connect Sydney and North Sydney. The subaqueous portion would be 1 416 ft. long, and the total length about $1\frac{1}{3}$ miles. The proposed section would be about 21 by $27\frac{1}{2}$ ft. inside, with a 16-ft. roadway and two $4\frac{1}{2}$-ft. walkways. The more recent recommendations, however, have been for a bridge.

Liverpool.—It has recently been proposed to connect Liverpool and Birkenhead by a tunnel under the Mersey for the use of vehicles and tram cars. As a successful passenger railway tunnel has been constructed under the river at this point, there would probably be no insurmountable physical difficulty in building a similar tunnel for highway purposes.

Oakland, Cal.—A tunnel has been proposed between Oakland and Alameda for a 20-ft. driveway and two 6-ft. suspended foot-walks.

METHODS OF CONSTRUCTION.

Existing highway tunnels have been constructed in soft ground by the shield method, by coffer-dam, or by sinking a metal form in a trench dredged from the surface, in which a lining of concrete was deposited; but, as other methods have been used successfully in build-

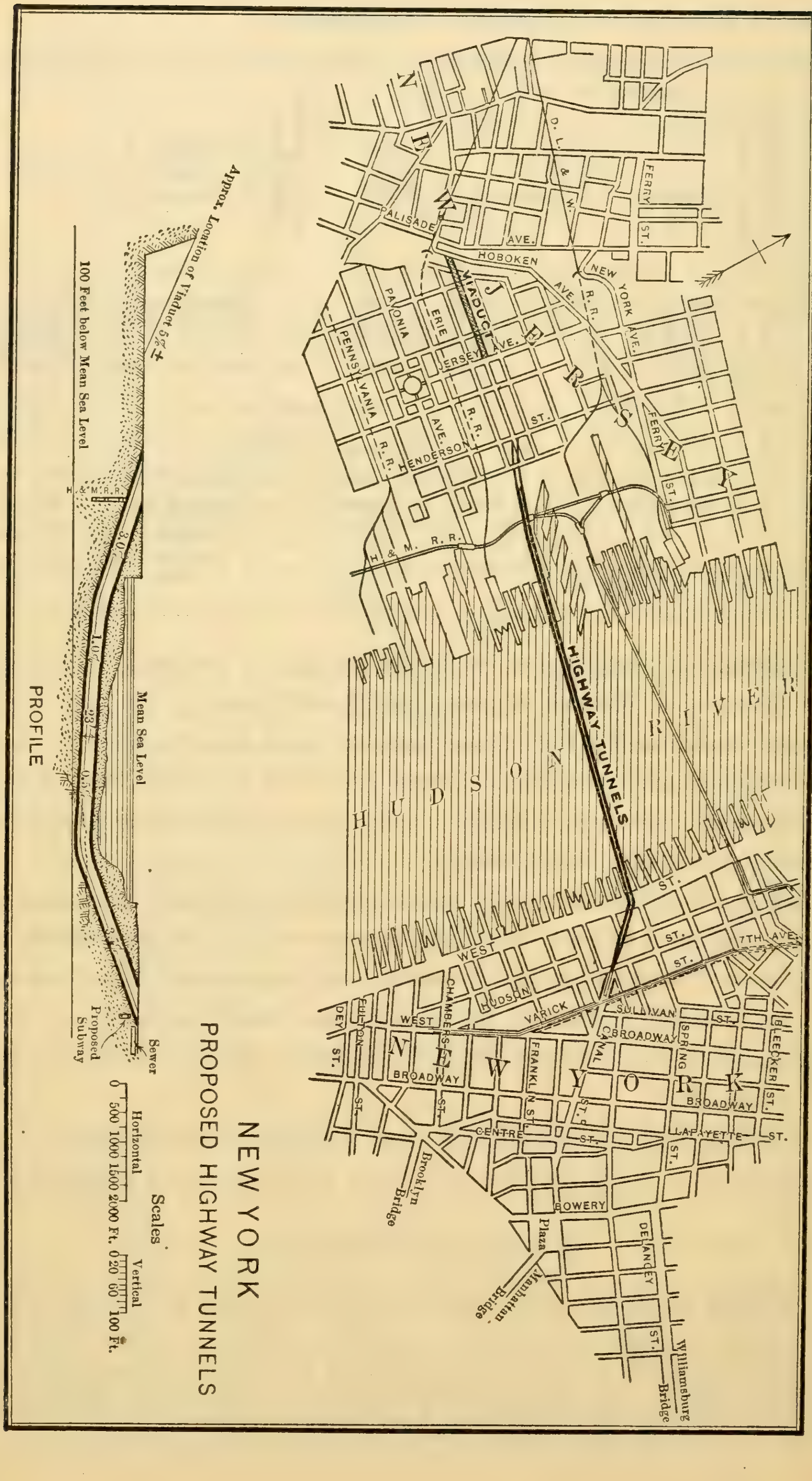

NEW YORK
PROPOSED HIGHWAY TUNNELS
PROFILE
Approx. Location of Viaduct
Mean Sea Level
100 Feet below Mean Sea Level
H. & M. R.R.
Proposed Subway
Sewer
Scales
Horizontal
0 500 1000 1500 2000 Ft.
Vertical
0 20 60 100 Ft.
NEW JERSEY
HUDSON RIVER
HIGHWAY TUNNELS
VIADUCT
NEW YORK
PALISADE AVE.
HOBOKEN AVE.
NEW YORK AVE.
JERSEY AVE.
HENDERSON ST.
PAVONIA AVE.
ERIE R.R.
PENNSYLVANIA R.R.
H. & M. R.R.
FERRY ST.
D. L. & W.
WEST ST.
CHAMBERS ST.
HUDSON ST.
VARICK ST.
7TH AVE.
SULLIVAN ST.
BROADWAY
SPRING ST.
BLEECKER ST.
LAFAYETTE ST.
FRANKLIN ST.
CENTRE ST.
BOWERY
DELANCEY ST.
DEY ST.
FULTON ST.
Brooklyn Bridge
Manhattan Bridge
Plaza
Williamsburg Bridge

FIG. 19.

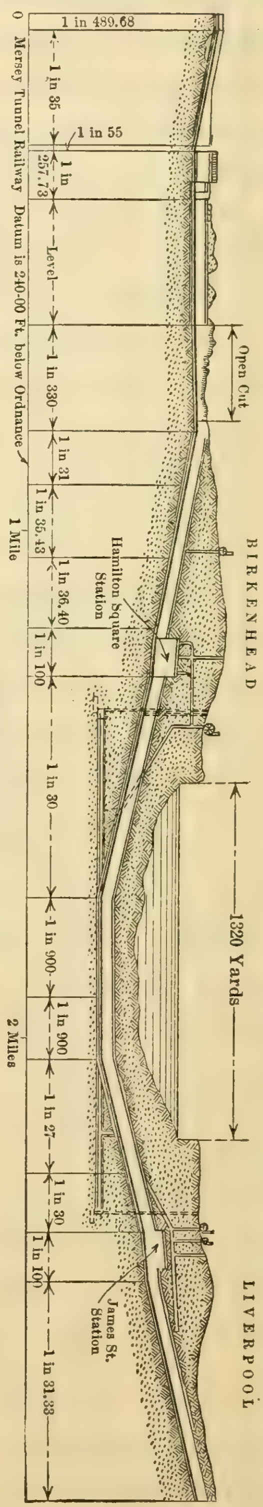
ing subaqueous tunnels for other purposes, they can be applied to the construction of highway tunnels where the conditions are favorable.

All the European examples are shield-driven. The first Thames Tunnel was driven without compressed air, but it was used on the Blackwall, Rotherhithe, Glasgow Harbor, and Hamburg Tunnels. In the United States, the Chicago tunnels were originally built within coffer-dams and afterward lowered by various means; in the La Salle Street Tunnel, a metal form was sunk from the surface.

Rock.—No subaqueous highway tunnels have been driven in rock formation, but several notable tunnels for other purposes have been constructed in rock, and there is no reason that they could not be built for highway purposes where the conditions are favorable. Both the Mersey and Severn Tunnels were driven in rock without the aid of compressed air. The Mersey Tunnel has an internal span of 26 ft. and a height of 23 ft. from the invert to the intrados. It is lined with brick masonry, and contains a double-track railway. It was opened on February 1st, 1886, and was first worked with steam locomotives and afterward by electricity. It is 3 960 ft. long between quay walls, and the total length of the line is 4.1 miles. It was excavated in sandstone, and the maximum depth of invert below mean high water is about 151 ft. Its cost, including equipment, was about £500 000 per mile.* Fig. 20 is a profile of this tunnel, and Fig. 22 is a cross-section.

Severn Tunnel.—The Severn Tunnel, Fig. 21, was excavated through conglomerate, carboniferous strata, sandstone, shale, and gravel. It is peculiar in that the width of the river at the point of crossing is 2¼ miles at high tide, but only 1 650 ft. at low water, the mean tidal range being 37 ft. Its total length is 4.36 miles. The maximum depth of the invert below high tide is 156 ft. The internal section of the tunnel is 20½ ft. high above rail level, and 26 ft. wide, and it is lined with brick masonry. The tunnel is notable on account of the difficulties encountered during construction. Water reached the workings through fissures and springs, the pressure from one spring being so great that it crushed the brickwork. This led to the sinking of an extra shaft at the site of the spring in which pumps were placed to remove the water and relieve the pressure. The total capacity of the pumping plant for the permanent drainage of the tunnel is

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXXVI, p. 40.



PROFILE, MERSEY TUNNEL

Scales

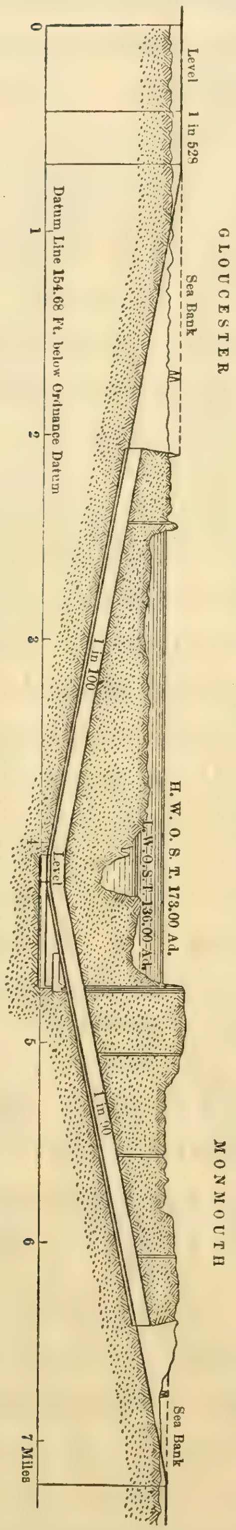
Horizontal

1000 750 500 250 0 1000 2000 Feet

Vertical

100 0 100 200 Feet

FIG. 20.



PROFILE, SEVERN TUNNEL

Scales

Horizontal

1 ¾ ½ ¼ 0 1 Mile

Vertical

100 50 0 100 200 Ft.

FIG. 21.

66 000 000 gal. per day, this being about twice the maximum quantity of drainage. The total cost of the tunnel was about £1 500 000.*

Various subaqueous tunnels have been constructed in rock for the conveyance of gas and water, some of these being at great depth, but the conditions of access are so different from those of a highway tunnel that they are scarcely comparable.

The conditions favorable to such tunnels in rock are where the strata are non-water-bearing, without faults or fissures, or where the depth of the water is beyond the maximum for work with air pressure. One advantage is that the tunnels can be given an economical cross-section more suited to their use than the circular form usually obtained with a shield.

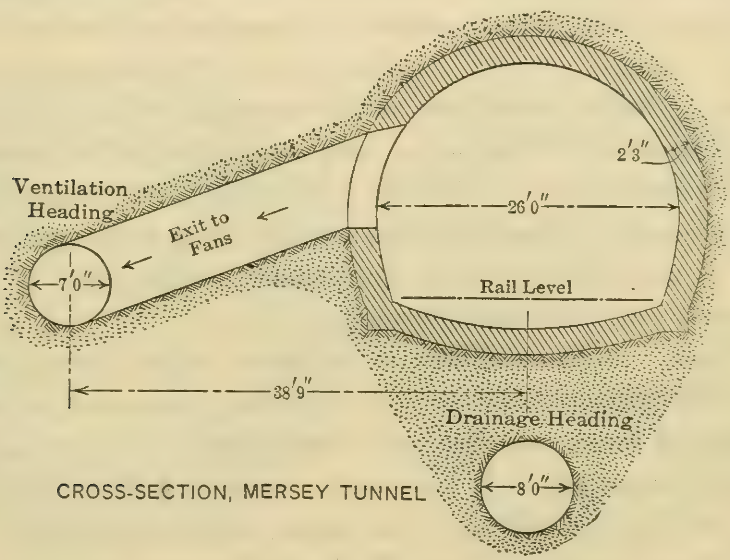


FIG. 22.

Shield Tunnels.—Four of the subaqueous highway tunnels have been driven in soft ground with a shield, and with the aid of compressed air. If the Thames Tunnel is included, there are five, but, in this latter case, compressed air was not used. Two of these tunnels (the Glasgow and the Hamburg), provide for a single line of vehicles in a tube, and two (the Blackwall and the Rotherhithe), provide two lines in opposite directions in a single tube, with a footway as well. The diameters of these tunnels range from 17 to 30 ft. A shield tunnel for a double line of electric railway has recently been completed under the Seine, in Paris, with a diameter of 25 ft. $6\frac{5}{32}$ in.

* "The Severn Tunnel," by T. A. Walker.

The conditions favoring the selection of the shield method of driving tunnels are: where the strata are of water-bearing silt, clay, sand, or gravel; where the bottom of the tunnel is not more than 100 ft. below the water surface; where a reasonable cover of material, between the top of the tunnel and the bed of the river, can be obtained; and where the channel of the waterway cannot be obstructed during construction. In suitable material very rapid progress can be made, the cast-iron lining facilitating rapid construction. On one tunnel of the Hudson and Manhattan Railroad, driven through Hudson River silt, a length of 72 ft. was built in one day.

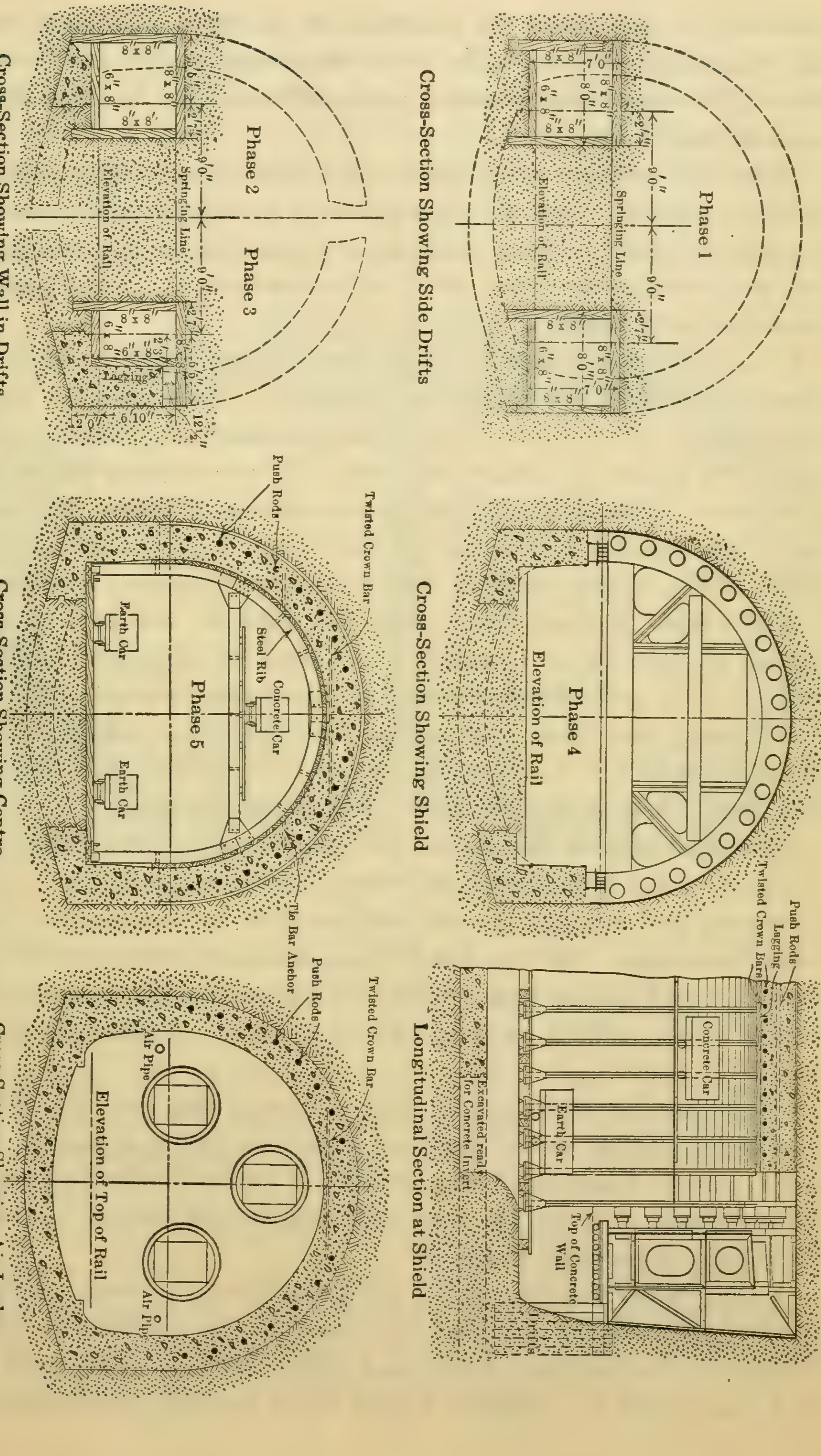
Shield construction is difficult when the lower portion of the tunnel is in rock and the upper portion is in silt or sand; it is also difficult in a porous soil with large boulders, as loosening or blasting the boulders is apt to cause a "blow", or loss of air. The defects of a porous soil, with a shallow cover, are often remedied by the use of a clay blanket dumped in the bed of the river on the line of the tunnel.

Shield tunneling for a highway is practically the same as for other purposes, the only difference being that, if more than one line of vehicles is to use a tube, it must be somewhat larger than is usual for such tunnels built for other purposes.

Practically all shield-driven subaqueous tunnels have been made circular, and although oval sections have been suggested, no such construction on a large scale has been carried out. The original Thames Tunnel was excavated in a rectangular section, within which the two brick arches were constructed, the space between the arches and the exterior of the excavation being filled with masonry backing.

Roof Shields.—No highway tunnels have been built with a roof shield, although this form of construction would give a more suitable section than a circular shield. A notable example of this form of tunneling is the East Boston Tunnel,* used by a double line of street cars. The internal span of this tunnel is 23.33 ft., with a height of 18 ft. above the rails. It was lined with concrete, which was about 3 ft. thick. The method of procedure, Fig. 23, was to excavate two small headings—one on each side—in which the sidewalls were constructed. This was followed by the shield, which rested on the sidewalls and was advanced by the jacks thrusting on push-bars embedded in the concrete. The excavation above the springing line and the

* Seventh Annual Report, Boston Transit Commission, August 15th, 1901.



EAST BOSTON TUNNEL. METHODS OF CONSTRUCTION.

Fig. 23.

concrete arch were under the protection of the shield. The final step was to complete the excavation below the springing line and lay the invert. The material encountered was a stiff blue clay in which there was very little water.

There are probably many places where this form of construction would be suitable, as all that is required is a material stable enough to enable the small advance headings to be constructed and the large face due to the width of the shield to be excavated, this being breast-boarded when necessary.

A variation of this method, which has not been tried, would be to construct the side-walls of concrete and then to build the roof, above the springing line, of cast iron resting on skewbacks, using a roof shield, and finally to complete the section by constructing the invert in concrete. This method might prove economical where the lower part of the tunnel is in rock and the upper part in soft ground, as tunneling with a circular shield is very expensive under these conditions.

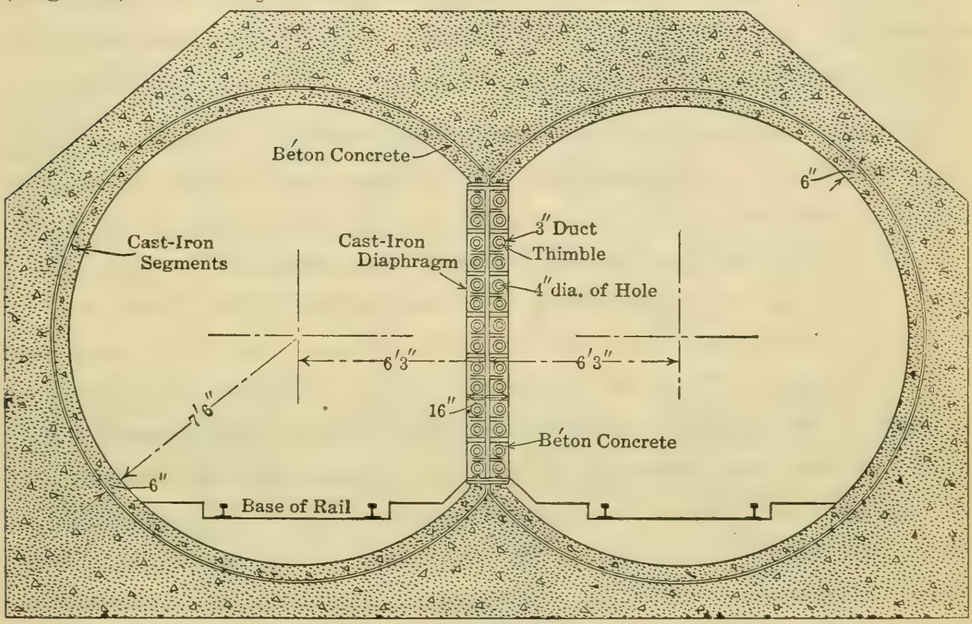
Coffer-Dam Methods.—The coffer-dam method was used in the original Chicago River Tunnels. It consisted of enclosing a portion of the river in coffer-dams, pumping out the water, excavating the enclosed space, and then constructing the subaqueous passage, after which the dam was removed and the channel restored; then similar steps were taken on the remaining sections of the river.

The advantages of this method are that the roof of the tunnel can be placed at the minimum depth, thus reducing the gradients of the approaches and the total descent and ascent to be overcome by those using the passage. The fact that it was necessary to lower the Chicago Tunnels would indicate that foresight should be used in fixing the elevations of the roofs of tunnels, built in this way, crossing navigable streams.

Conditions favorable to this form of construction are: the ability to close temporarily part of the river channel at a time, a more or less impervious river bed which will prevent the water from leaking under the coffer-dams and into the space excavated for the tunnels, a stream not subject to excessive floods, etc.

Harlem River Crossing, New York Rapid Transit Subway.—A variation of the coffer-dam method was developed in the Harlem River crossing of the first New York rapid transit subway, which could be used equally well for a highway tunnel. This crossing was constructed

in two sections, one-half the width of the river being closed at one time. The plan of construction was devised by the firm of McMullen and McBean, the contractors, and as the methods used in the south-westerly half of the river differed somewhat from those in the north-easterly half, the former will be described first. In section, the tunnels consist of two cast-iron segments of a circle with a vertical diaphragm separating them, the whole being encased in concrete. (Fig. 24.)



CROSS-SECTION, HARLEM RIVER TUNNEL

FIG. 24.

The successive steps were as follows:

1. The material on the line of the tunnel was excavated to somewhat below the springing line by dredging from the surface.

2. Four longitudinal rows of piles were driven. These were 8 ft. apart longitudinally and 6 ft. 4 in. transversely. They were then cut off under water so that their tops were about 11 ft. above the axis of the tunnel.

3. The sides of the excavation were then enclosed by sheet-piling, consisting of 12 by 12-in. long-leaf yellow pine timbers, about 65 ft. long, bolted together into units 3 ft. wide. This sheeting was guided by a pile platform along each side and by a timber frame. These piles were then cut off under water by a circular saw.

4. On top of the four rows of bearing piles and the sheet-piles, was then sunk a timber deck consisting of three transverse layers of 12 by 12-in. timbers and two longitudinal layers of 2-in. plank. After this was sunk accurately in place, it was covered with a thickness of about 5 ft. of earth and mud.

5. Several vertical shafts were placed, passing through this deck, and, after placing bulkheads at the ends and air locks, compressed air was turned on and the excavation was completed, the iron lining was erected, and the concrete backing placed. The bearing piles were cut off and removed as the work progressed.

In the construction of the northeasterly section, instead of building a timber air floor resting on sheet-piling, inside which the excavation was completed and the lining placed, the upper half of the lining was erected inside of a floating box and then sunk on the sheet-piling and utilized as an air floor for the construction of the lower half of the work. This floor was built in three sections—two 90 ft. long and one 84 ft. long. When the upper half of the structure had been constructed in the floating box above its final position, the suspender rods were attached, and sufficient water was pumped into the box to sink it. It was then pulled out lengthwise from under the roof, and the latter was sunk into position, the end joint being made by a diver. After being sunk to position, compressed air was installed, the excavation was completed, and the lower half of the lining was constructed (Fig. 25).

A design for the application of this method to a highway tunnel was prepared by the Carnegie Steel Company and presented in a paper by R. B. Woodworth, M. Am. Soc. C. E., before the Railway Club of Pittsburgh.* (Fig. 26.) This plan contemplates a roadway 60 ft. wide, with two electric car tracks, the floor and side-walks being of concrete and the roof of steel embedded in concrete.

Another design for a highway tunnel by this system, having two roadways, each 19 ft. 6 in. in width, was shown before the Hoboken Board of Trade on February 4th, 1913. (Fig. 27.)

Detroit Tunnel.—The method used in constructing the Detroit River Tunnel has been described so thoroughly in the paper† by W. S.

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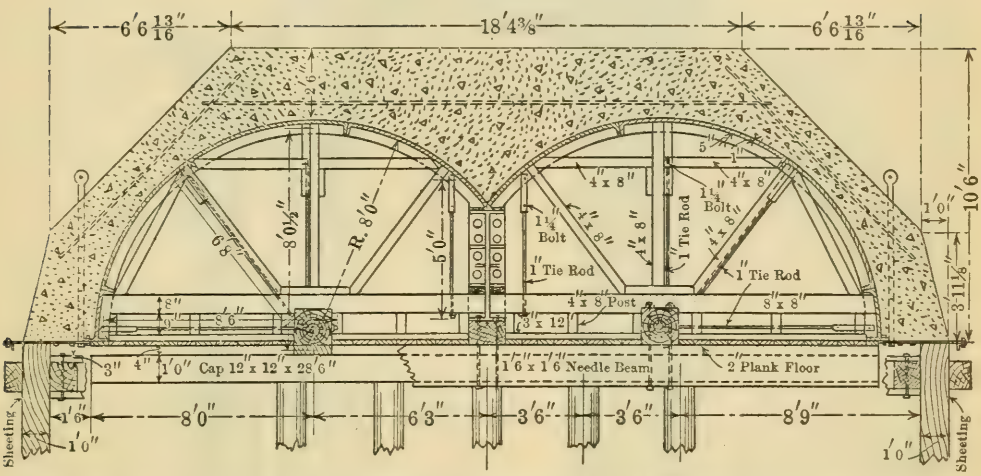
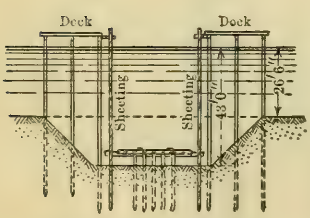
* *Official Proceedings*, Railway Club of Pittsburgh, December 22d, 1909.

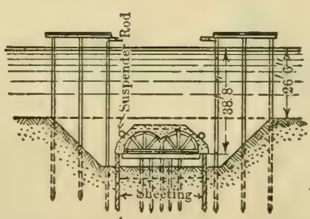
† *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 288.



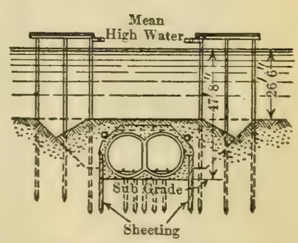
CROSS-SECTION, SHOWING THE IRON AND CONCRETE ROOF CONSTRUCTION IN PLACE SUPPORTED BY THE SIDE-SHEETING AND THE CENTER PILES.



Tunnel Trench, Docks and Submerged Frame Resting on Pile Caps.

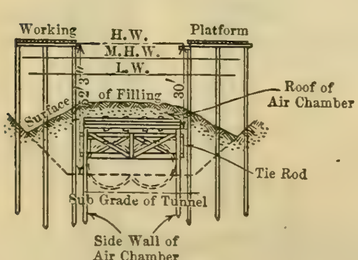


Sheeting Cut-off and Upper Part of Tunnel Shell in Position, Forming the Roof and Side-walls of a Working Chamber.



Section of Completed Tunnel.

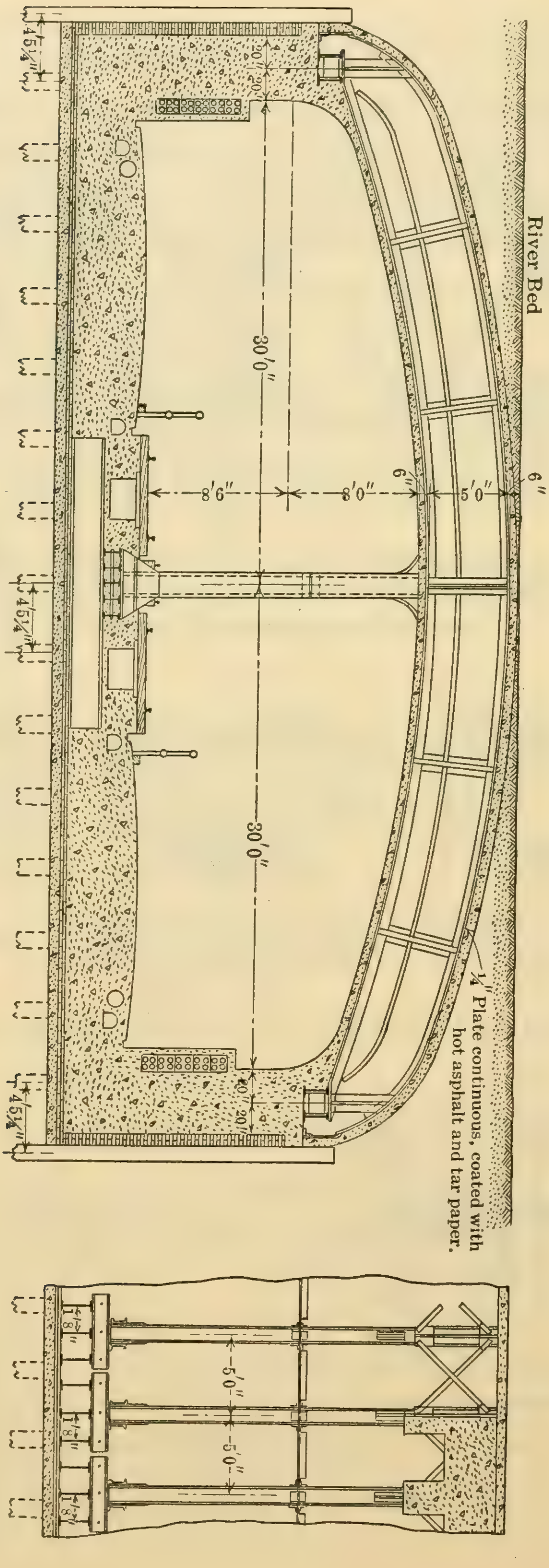
METHOD ADOPTED IN BUILDING EASTERN HALF OF TUNNEL.



METHOD ADOPTED IN BUILDING THE FIRST, OR WESTERN HALF OF THE TUNNEL.

HARLEM RIVER TUNNEL.
METHODS OF CONSTRUCTION.

FIG. 25.

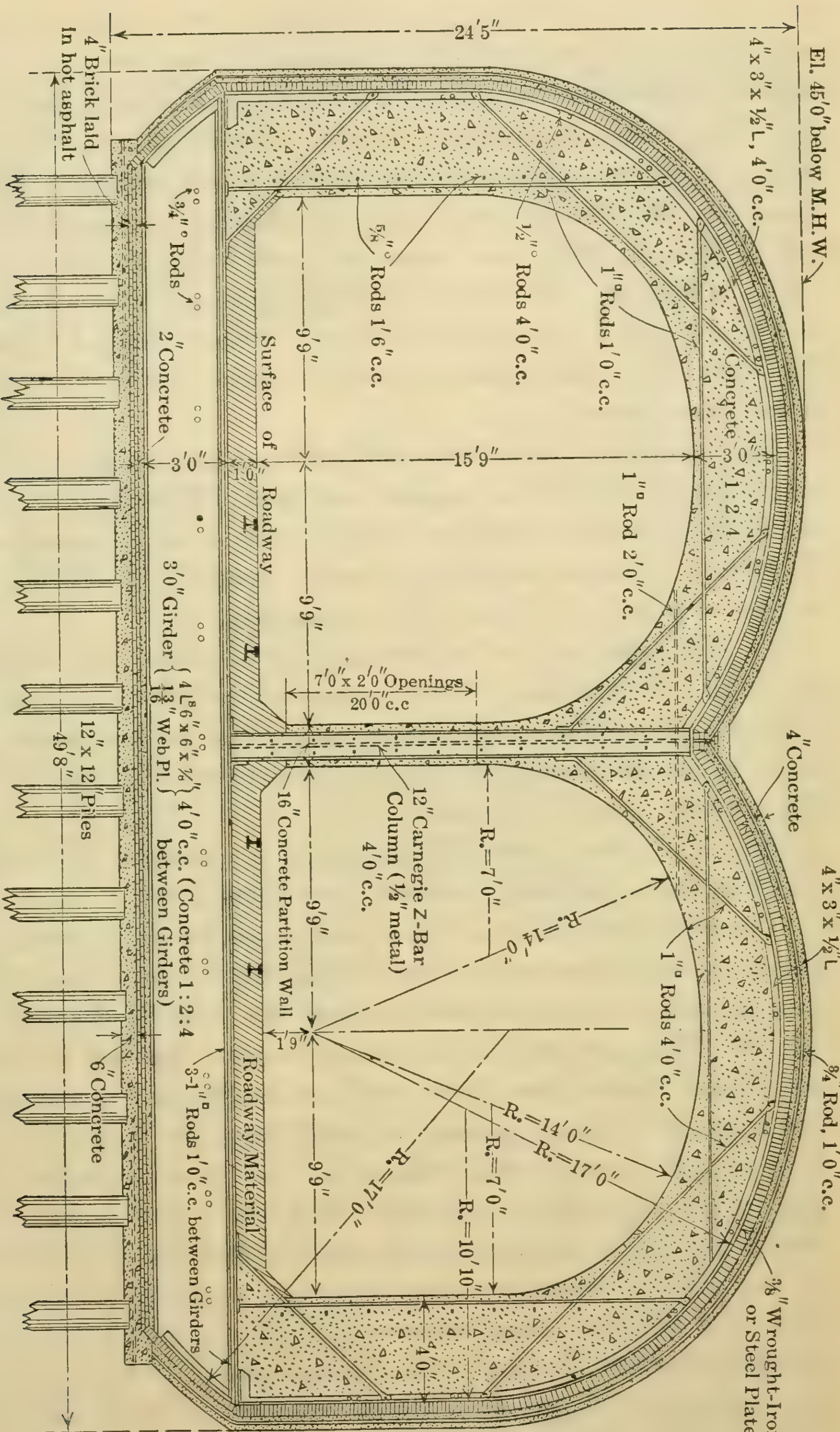


CROSS-SECTION

PROPOSED HIGHWAY TUNNEL.

Fig. 26.

LONGITUDINAL SECTION



CROSS-SECTION, PROPOSED HIGHWAY TUNNEL

Fig. 27.

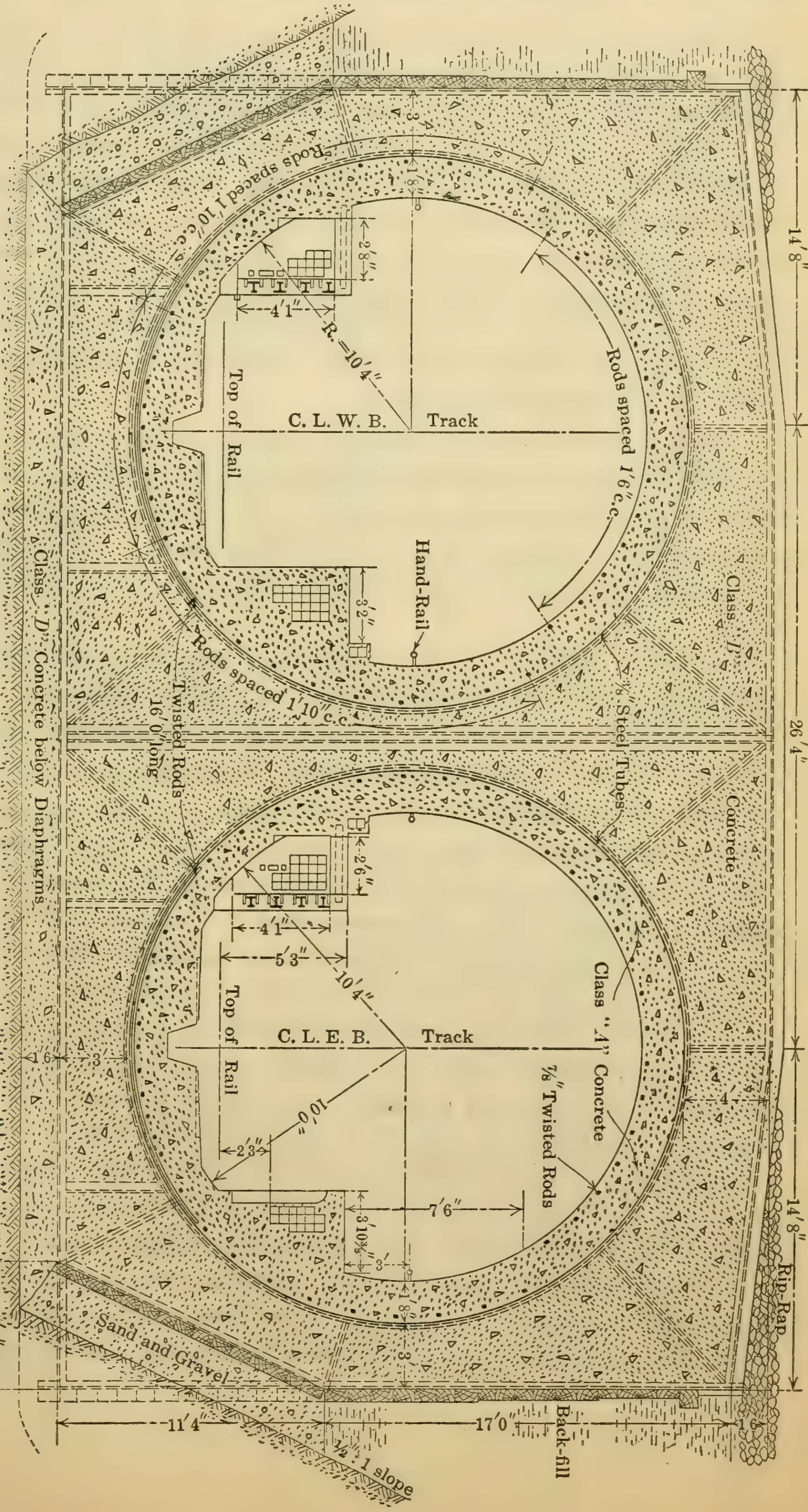
Kinnear, M. Am. Soc. C. E., and in the paper* by W. J. Wilgus, M. Am. Soc. C. E., that but little comment is necessary. Briefly, the method consisted of dredging a trench along the line of the structure into which were sunk sections of steel tubes which acted as forms to limit the concrete deposited from barges through tremies. These sections were 262 ft. 6 in. long. After the sinking, concreting, and joining of sections, they were unwatered and a lining of concrete was added. (Fig. 28.) The roof of this structure is 41 ft. below the surface of the water, and for some distance is above the bed of the river. The approaches were constructed with a shield, partly with and partly without compressed air.

The conditions favorable to this form of construction are: a material that can readily be dredged and that will enable the slopes of the dredged trench to stand at a fairly steep slope; a modest current, not subject to sudden fluctuations; and a channel, the obstruction of a portion of which at a time will be permitted.

Its advantages are that a structure can be placed at a higher level than with a driven tunnel, with a consequent reduction in the gradients of approach, and the section can be given a shape conforming to the use to which the tunnel is to be put, without being limited to the circular section usual with a shield.

Caisson Method.—Several sections of the tunnels of the Hudson and Manhattan Railroad Company were constructed in short lengths of reinforced concrete, which were sunk as pneumatic caissons. Three special switch enlargements were built in this way in Jersey City, and in these cases the tunnel itself formed the working chamber of the caisson. The approaches to the Church Street Terminal in Cortlandt and Fulton Streets, New York, were also constructed in this way, but here a special working chamber was built below the floor of the tunnel. However, the most notable example of this form of construction is that of the Metropolitan Railway, in Paris, which crosses the two arms of the Seine between Porte de Clignancourt and Porte d'Orleans.† In this case the larger arm was crossed by using three caissons, 118, 125.9, and 141.7 ft. long, respectively, and the smaller arm by two caissons, each 65.9 ft. long. Two stations under the land nearby were also constructed in this way. These caissons were sunk with a space

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXV, p. 2.
† "Les Travaux Chemin de Fer Métropolitain Municipal de Paris à la Traversée de la Seine," by L. Biette.



CROSS-SECTION, DETROIT RIVER TUNNEL

FIG. 28.

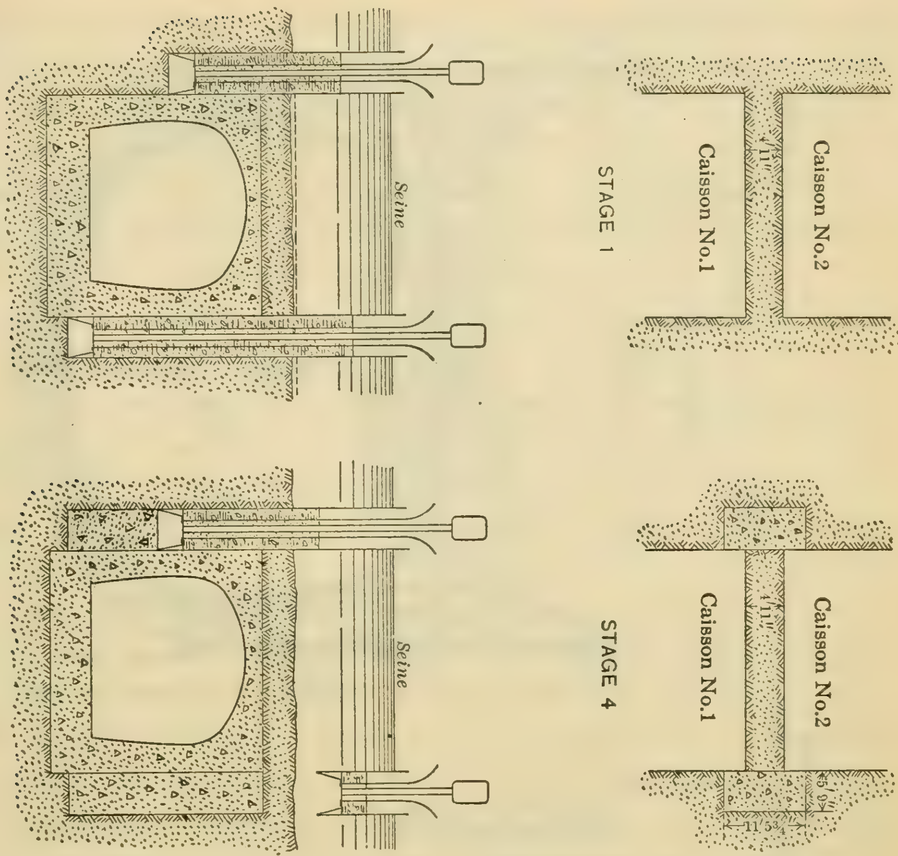
of 4.92 ft. between their ends. These gaps were closed by sinking caissons at each side, which were withdrawn after the excavation was completed, and the spaces were filled with concrete. In like manner, the roof was sealed by sinking a caisson spanning the other two, inside of which a concrete slab was built. (Fig. 29.) Each caisson consisted of a framework of structural steel, embedded in concrete, with a working chamber below the floor of the tunnel. (Fig. 30.) They were erected on the shore, launched, floated into position, and sunk. A portion of the material in the river bed along the line of the tunnel was removed by dredging before sinking the caissons. A cast-iron lining was placed within the structural steel framework.

This is a reliable method, applicable where conditions do not permit sufficient cover for the shield method, and by it the roof of the structure may project above the bed of the river if the navigation and discharge of the stream will not be obstructed thereby. In applying it to a highway tunnel, a section suitable for this purpose can be obtained.

Favorable conditions for its use involve a modest current and the opportunity to obstruct a portion of the channel at a time.

The objections are that it is probably the most expensive method of any on account of the structural steel, the large quantity of concrete, the cast-iron lining, the cost of excavating under compressed air, and the extra excavation due to the working chamber being below the floor of the structure, as well as the cost of making the joints between adjoining caissons. It would appear that the method followed at Paris could be materially modified, with great saving in cost: first, by making the caissons of reinforced concrete, without the structural-steel framework; second, by making the interior of the structure itself the working chamber, with such temporary or permanent transverse members at the floor level as might be necessary, between which the excavation could be made; third, by eliminating the cast-iron lining as being unnecessary; and, fourth, by sinking the caissons with a gap of from 6 to 12 in. between them, instead of nearly 5 ft., so as to enable the joint to be made in a much simpler and cheaper way.

Freezing Method.—The construction of subaqueous tunnels by the freezing method has been proposed. It has been used for numerous shafts and excavations in wet ground for foundations, and in driving



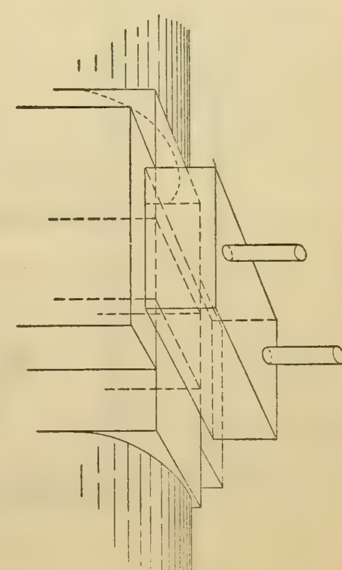
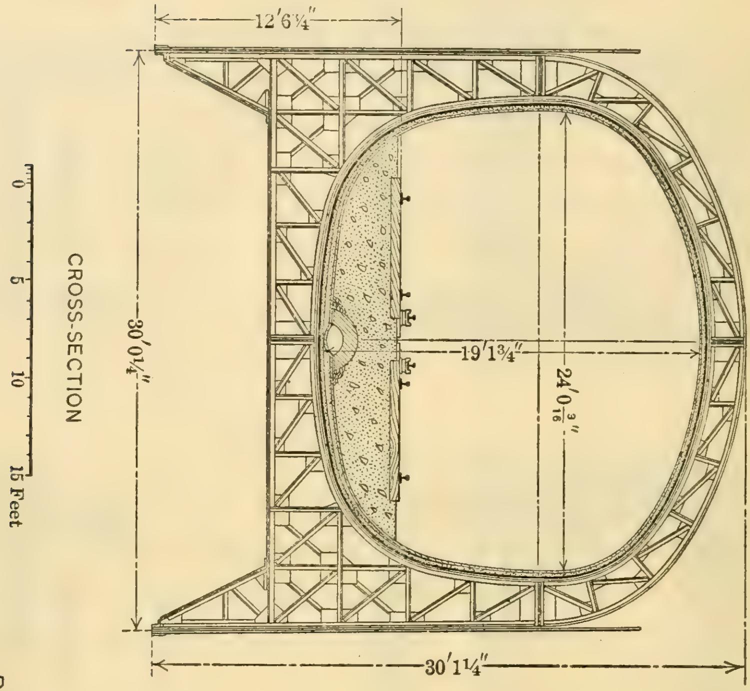
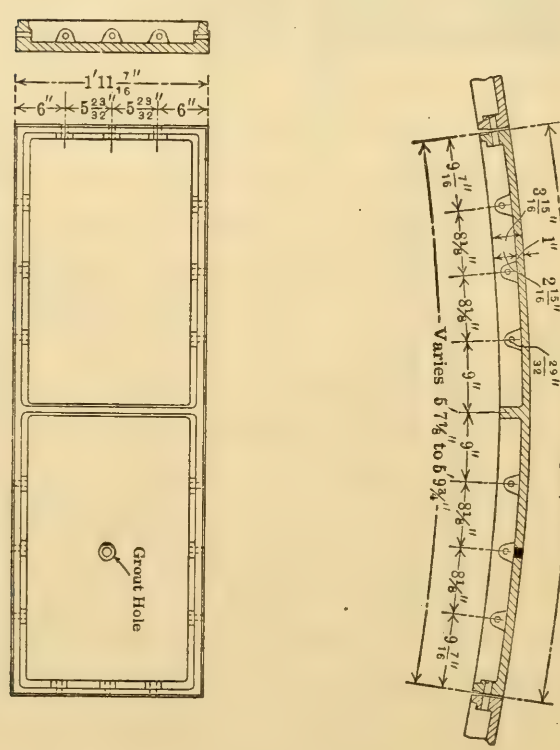


FIG. 29.

PARIS
DIAGRAM SHOWING METHOD OF
SEALING JOINTS BETWEEN CAISSONS



CROSS-SECTION



DETAIL OF SEGMENT

PARIS CAISSONS

Fig. 30.

a tunnel in wet ground in Stockholm.* A short experimental tunnel was built by this method by the Pennsylvania Railroad Company under the East River at New York, with a view to its use in the construction of the tunnel connection to the Long Island Railroad, in case the shield method should have proved unsuccessful.† It was found, however, that the rate of freezing was so slow, and the shield proved so successful, that no practical use of that method was made.

It was also used by the contractor, in an experimental way, on the East River Tunnel of the New York City subway system, at a point where he had not been able to drive the tunnel to the correct alignment and grade.‡ An attempt was made to freeze an annular space surrounding the tunnel and then to reconstruct the lining in its true position, but the method was abandoned and the tunnel was rebuilt by a different method.

The freezing method was also used on a portion of the Metropolitan Railway crossing the Seine at Paris. In building by caissons the part of the line previously referred to, it was found that some other method had to be used at the point where the line crossed under the structure and roadway of the Orleans Railway (which is built along the quay), which would not interfere with the railway operation, and the freezing method was tried.§ It was necessary to connect the last caisson in the river with the first caisson on the land, a distance of about 200 ft. This work was executed by freezing the surrounding soil by circulating a solution of calcium chlorate through pipes placed in holes bored horizontally into the soil. In clay and calcareous earth this method was fairly successful, but, when alluvial soil with boulders was reached, the boring of the holes was so difficult that the method had to be abandoned. The work was then continued by ordinary soft ground methods, with timber bracing and lagging, until a point beyond the structures of the Orleans Railway was reached. The freezing process was then resumed, using vertical tubes for the circulating medium. The connection with the caisson construction was nearly made when the water broke through the frozen material and flooded the tunnel. A bulkhead was placed across the tunnel, which was then unwatered, and the connection with the caisson was finally made by this method.

* *Transactions*, Am. Soc. C. E., Vol. LII, p. 365.

† *Transactions*, Am. Soc. C. E., Vol. LXVIII, p. 24.

‡ *Engineering Record*, December 15th, 1906.

§ *The Engineer*, July 1st, 1910.

Notwithstanding the general lack of success with this method, conditions may arise where it would be the most suitable, and it should be considered as one of the resources of the engineer.

Submerged Bridges.—Various projects for submerged tubes, or bridges, have been proposed for crossing bodies of water, the object being to avoid excessive depth and the cost of excavation. Some of these plans are very ingenious, but none has ever been carried out, although many patents on these schemes have been issued.

Lining.

The type of lining is largely dependent on the method of construction. With a circular shield, cast-iron rings with internal flanges are generally used. Cast-steel rings have been used at points where special strength was desired. Considerable thought has been devoted to the substitution of rolled, or pressed, steel segments for the cast iron, but they have been tried only in an experimental way in America. The idea has been that, for a tunnel which must have a lining of masonry, the metal lining should only be considered as a temporary sheeting to enable the permanent lining to be constructed inside thereof and as a water-proofing envelope, and that it, therefore, should be as cheap and light as possible. The tunnel under the Elbe at Hamburg was lined with structural steel with a concrete lining.

With the same object in view, several water and sewerage tunnels have been constructed with a lining of wooden segments, inside of which the concrete lining was built. The approaches to the Detroit River Tunnel were lined with wood in this way.

The Great Northern and City Railway Tunnel, London, was lined with cast iron as the shield was driven, after which the segments in the lower half of the tunnel were removed and replaced with brick masonry; the iron thus saved being used over again.

Brunel's Thames Tunnel was lined with brick masonry, as was a considerable portion of the original Hudson Tunnel, but the latter was not built with a shield.

The original Chicago River Tunnels were also of brick laid in cement, with the outer courses laid in asphalt.

The East Boston Tunnel was lined with concrete and built with a roof shield.

Concrete lining is also essential to the methods used at Detroit and in the subway tunnel under the Harlem River, New York.

Concrete blocks, in lieu of cast-iron lining, for shield-driven tunnels have been proposed and patented by John F. O'Rourke, M. Am. Soc. C. E. (Fig. 31.)*

In a favorable stratum, a subaqueous highway tunnel might be built without any artificial lining at all, but, for sanitary reasons and for ease in illumination, it is doubtful if this would be advisable. The East River Gas Tunnel, at Seventy-first Street, New York City, is about 100 ft. below the surface of the water and is only lined for a short distance in the vicinity of fissures in the rock.

As to the relative advantages of the different linings, cast iron is especially suited to shield construction, and has the advantage of being readily made water-tight, together with its ability to resist the effects of corrosion. For a highway tunnel, it has the disadvantage of being unsuited to any but a circular section. Concrete is readily adapted to a section of any shape. When used with a shield, it must have metal members embedded in it to take the thrust of the shield jacks. When in water-bearing strata under great head, it is difficult to make it entirely water-proof in itself, and it is also difficult to apply an extraneous envelope of water-proofing. A warning should also be sounded in regard to its use in strata containing acid or alkali waters, which rapidly disintegrate concrete. Such waters, though not usual, are not rare. In the first attempt to build the Detroit River Tunnel, sulphur water and sulphur gas were found in large quantities, and sulphuric acid has been encountered on other underground works, so that analyses and tests of underground waters should be made on all tunnel work.

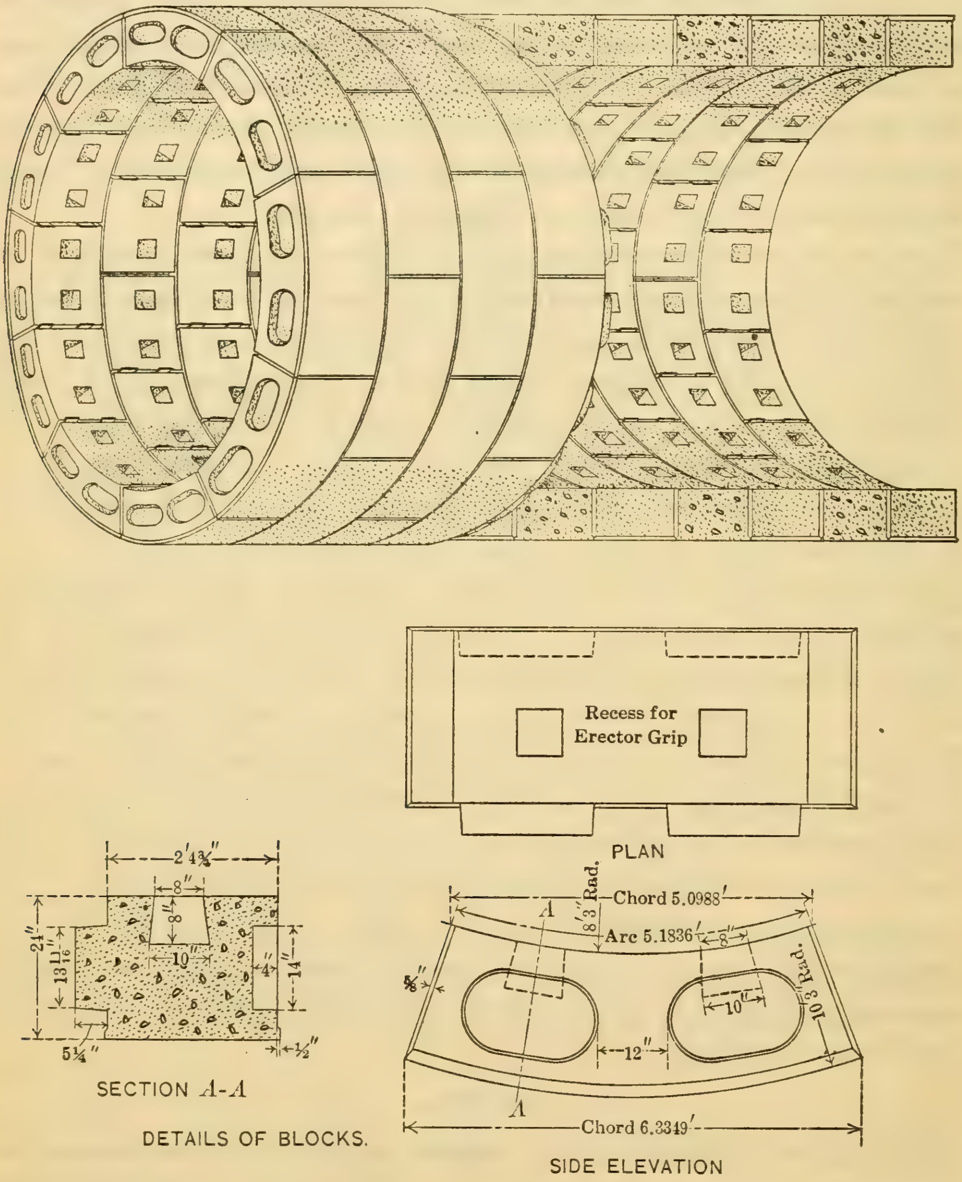
An electrical survey should also be made, in order to determine the probable effect of such a tunnel in forming a new path for stray electric currents, and steps should be taken to prevent damage to the structure from electrolysis.

Section.

The shape of the section is largely determined by the method of construction, rather than by the use to which the tunnel is to be put. Shield tunnels have generally been circular, and this section is not generally well adapted to highway purposes, a semicircular, or horseshoe section being more suitable. The Blackwall Tunnel has a 16-ft. roadway and two 3 ft. 1½-in. footwalks, and the Rotherhithe Tunnel

* *Engineering News*, November 28th, 1912.

has a 16-ft. roadway and two 4 ft. 8½-in. footwalks. The section indicated in Fig. 32 is suggested as offering a more economical use of the available space, if a footwalk is essential, as it provides a 12-ft. foot-



CONCRETE TUNNEL LINING

FIG. 31.

walk and 20-ft. roadway. In tunnels more than 1 mile in length, it is doubtful if a sufficient number of persons would use the footwalk to warrant its construction, particularly where facilities are provided for crossing in electric cars in the same, or another, tunnel. If the

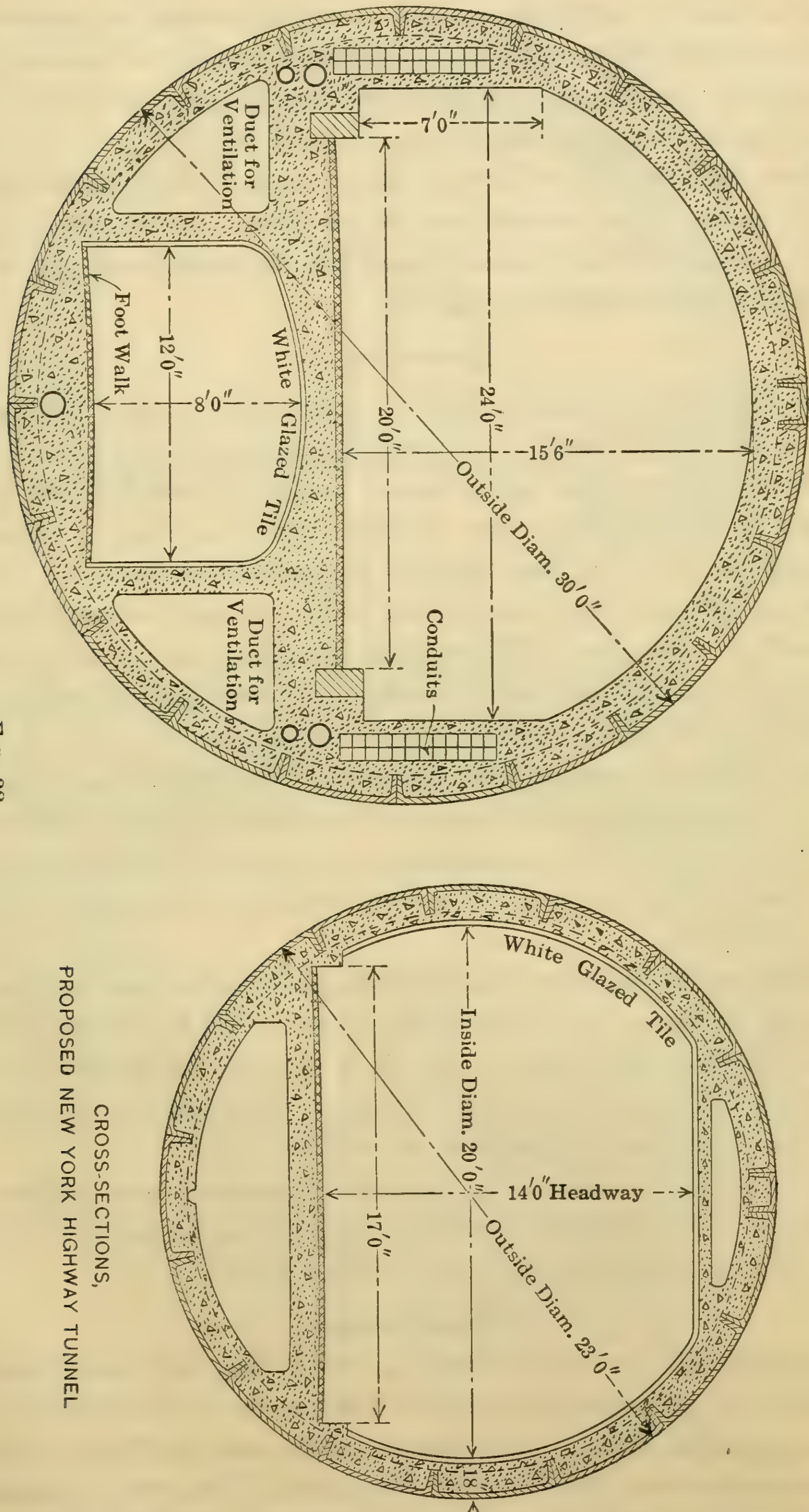


FIG. 32.

CROSS-SECTIONS,
PROPOSED NEW YORK HIGHWAY TUNNEL

tunnel is made of sufficient size for only the roadway, a material reduction in area can be made, as a 17-ft. roadway can be obtained in a tunnel 23 ft. in diameter, as shown by Fig. 32, instead of the 30-ft. diameter adopted for the Rotherhithe Tunnel.

Where the Detroit or Harlem River methods of construction are used, a more suitable section can be obtained. A suggested section for the proposed Sydney Harbor Tunnel, to be constructed by the Detroit method, is indicated by Fig. 33.

Approaches.

The Rotherhithe and Blackwall Tunnels have inclined approaches, with gradients of 1 in 36. The gradients on the Chicago Tunnel are as high as 10%, but these are no longer used by general highway traffic, but only by electric street cars. In determining a proper gradient for an approach for a highway tunnel, the prevailing gradients on the adjoining highways which would have to be traversed to reach the tunnel should be considered. Where these are high, there is no objection to using an equally high rate in the tunnel approaches; otherwise, they should be as low as the conditions and cost permit.

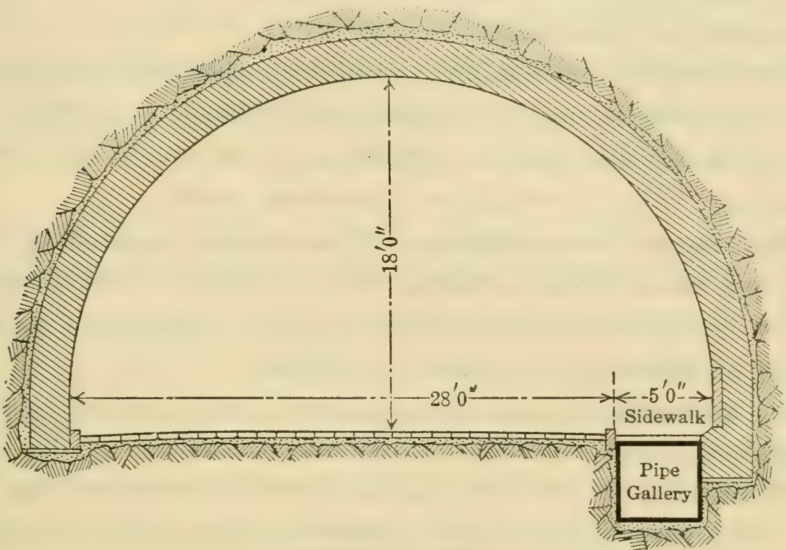
For structural reasons, the approaches to a bridge are generally made on a prolongation of the axis of the main span, but with a tunnel this is not necessary, and they may be curved or at any convenient angle with the main tunnel, and, in fact, may be made parallel with the river, if need be.

The Glasgow and Hamburg Tunnels have no inclined approaches, but access is obtained by vertical lifts in shafts. The Glasgow Tunnel has six elevators for vehicles, and has a vertical lift of about 65 ft.; the Hamburg Tunnel has four elevators for vehicles and two for passengers, and has a vertical lift of 78 ft.

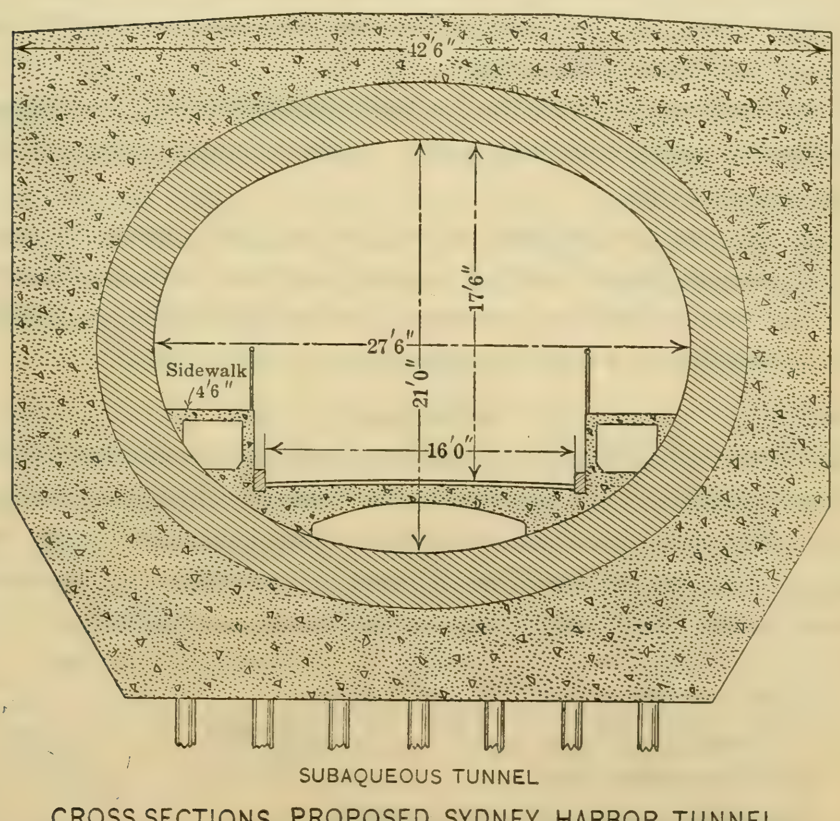
A combination of these two methods of approach may be made by using shafts with elevators for access to and from points at the shore, with inclined approaches for points farther inland.

Water-Proofing.

Tunnels with a cast-iron lining can readily be made water-tight by caulking the space provided in the lining at the joints, which is one advantage of this form of construction. Also, with the Detroit method, the sheet-steel tube, around which the external concrete is deposited, can be caulked and will keep the tunnels tight. Tunnels sunk by the



APPROACH TUNNEL
(ROCK)



SUBAQUEOUS TUNNEL

CROSS-SECTIONS, PROPOSED SYDNEY HARBOR TUNNEL

FIG. 33.

caisson process can have an inner or outer shell which can be caulked and will serve as a water-tight envelope.

Masonry under-river crossings, built by the open coffer-dam method, can be water-proofed by the membranous method, using felt, duck, canvas, or burlap, and pitch or asphaltum, or an enveloping course of brick masonry laid in asphaltic or bituminous mastic.

With a concrete lining laid in a driven tunnel, satisfactory water-proofing is a more difficult problem. If the tunnel is constructed with the aid of compressed air, the application of a bituminous compound that must be heated to be applied, is difficult.

In the approaches to the Detroit River Tunnel, which were lined with concrete, the water-proofing consisted of from 3 to 11 plies of felt and pitch applied to the wooden lagging before the concrete was placed.

The East Boston Tunnel was constructed in a more or less impervious clay, and no water-proofing was used.

The liberal use of grout at a greater pressure than that due to the hydrostatic head, will add greatly to the imperviousness of the concrete and surrounding material.

In the reconstruction of the La Salle Street Tunnel, in Chicago, a water-proofing compound was added to the concrete with a view to making it water-tight.

Some concrete-lined electric railway tunnels have been water-proofed with an internal plaster coat of cement to which water-proofing compound had been added, but, owing to contraction cracks, it is difficult to get an absolutely water-tight structure by this method.

Ventilation.

Thus far the ventilation of highway tunnels has not proved to be a serious problem. In the Blackwall Tunnel ventilation ducts and provision for fans were made, but the fans have never been found necessary. In the Rotherhithe Tunnel no provision for ventilation has been made. The traffic in this tunnel has been as follows:

	1911.	1912.
Total number of vehicles, including motors	896 629	973 336
Motor vehicles only	21 008	26 998
Maximum number of vehicles passing through the tunnel in one hour		325

Of the latter, 10 were motor-driven.

The figures for the Blackwall Tunnel are somewhat similar.

Although, under the foregoing conditions, artificial ventilation has not proved necessary, it is probable that, with the increasing use of motor-driven vehicles having internal-combustion engines, provision for the mechanical ventilation of long highway tunnels will have to be made.

The exhaust gases given off by a gasolene motor consist of carbon monoxide, carbon dioxide, and free oxygen. Of these, the most poisonous is the carbon monoxide, and if sufficient fresh air is provided to dilute this to within safe limits, the other gases will not cause any trouble. In addition to this, sufficient air must be provided to furnish the oxygen necessary for combustion. The character and quality of these gases has been determined very accurately in some recent tests* which indicate that, with a 6-cylinder, 48-h.p. motor, the quantity of air required to dilute the gases given off, within safe limits, would be about 8 000 cu. ft. per sec. when in first gear, 4 000 cu. ft. per sec., when on second gear, and 2 000 cu. ft. per sec., when on third gear. Based on these figures, it will be found that the ventilation of such a tunnel, even assuming that motor cars follow each other as closely as safety permits, can be readily accomplished. It can be effected by withdrawing air from the center of the tunnel through conduits under the floor, as in the Blackwall Tunnel, or above the clearance line, as in the East Boston Tunnel, or through separate ventilating passages, as in the Mersey Tunnel.

Drainage.

The Severn and Mersey Tunnels are both drained by letting the water leaking into them flow in separate tunnels, constructed for the purpose, from the lowest points in the main tunnels to shafts on the shore, where it is removed by pumping. In both these cases the tunnels were constructed in rock and lined with brick masonry, and the leakage was very large, but in more recent tunnels, lined with iron, steel, or concrete, the leakage is so small† that such special drainage headings are hardly necessary. In these latter tunnels the water is removed by pumping from the sumps through pipes laid along the sides of the tunnel to the shore.

* "A Comprehensive Motor Test," by Herbert Chase, *Transactions*, Soc. of Automobile Engineers, 1912, p. 40.

† Detroit Tunnel, *Transactions*, Am. Soc. C. E., Vol. LXXIV, p. 349; Hudson Tunnel, *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXI, p. 169.

The art of boring large-sized holes in any direction has developed to such an extent that there is no reason why a hole cannot be bored from the sump upward through the overlying material to the bed of the river through which to discharge the drainage and thus save the long length of pipe to shore and the increased friction head.

Conclusions.

In this paper the writer has endeavored to assemble the published facts relative to all the subaqueous highway tunnels in the world, as well as of subaqueous tunnels built for other purposes, where the methods of design or construction would have an application for highway purposes. The paper, therefore, is largely a compilation, the information having been obtained from the *Transactions* of this and other engineering societies, and from the technical press. Credit for this information is generally given, but in a number of cases where the information came from various sources, the authority is not stated, and the writer wishes to take this opportunity to express his indebtedness to those who have written on this subject.

The need for improved methods of vehicular communication around harbors is manifest, and it would seem that the furnishing of this by highway tunnels has not received the attention it should. This paper, therefore, has been prepared in the hope that it will develop a discussion which will stimulate attention to tunneling for this purpose.

Commercial transportation was formerly largely conducted on the highways, and the world will shortly complete a century of railway transportation, one of the consequences of which was neglect of the highways, but the present tendency is to return to the highway for short-haul transportation, largely owing to the development of the motor vehicle for pleasure and business, which is leading to a revival in the study of road construction to meet new conditions, and highway tunnels, in many localities, are a necessary link to make this system of communication continuous.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

DISCUSSION ON VALUATION FOR THE PURPOSE OF RATE-MAKING.*

By Messrs. J. N. Dodd, Morris Knowles, M. O. Leighton, J. P. Snow, Arthur L. Mills, J. M. Schreiber, J. P. Newell, Gardner S. Williams, George F. Swain, J. W. Ledoux, Albin G. Nicolaysen, J. B. Lippincott, Alfred Noble, M. L. Byers, and Frederic P. Stearns.

J. N. Dodd, Esq.—The general principle involved in valuation for rate-making has been stated very clearly in the report, namely, that the primary requirement is that justice be done both to the owner of the property and to the public, and that the owner is "entitled to earn a fair return upon a fair value of the property utilized in or reasonably necessary to the service". This has been settled by the Courts, and no dissent can be taken from such ruling. On the other hand, as pointed out by the Committee, no complete or definite rule has been laid down for the determination of the fair value. Mr. Dodd.

It is becoming generally agreed that, as regards what are called public utilities, the public, as a rule, is better served by a monopoly than by competition. One gas company can manufacture and distribute gas more cheaply than six companies. The distribution by one company can be made more satisfactory, both to the consumer and to the public at large, than by six. For example, the disturbance due to tearing up the streets is far less when one company produces and distributes than when there are six.

On the other hand, the dangers of a monopoly are well realized. One of the functions of the Legislature, now usually delegated to the public service commission, is to guard against such dangers and to protect the consumer against unreasonable demands on the part of public utilities.

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* Continued from April, 1914, *Proceedings*.

Mr. Dodd. For future and for recent public service properties, the Committee favors the actual cost as the fair value. This is apparently the value to be given to the property for all time to come. For old public service properties, the Committee favors the cost of reproduction less appreciation of the identical plant at present-day prices of labor and material, mainly because it is claimed that it is impossible to determine the actual original cost of such properties.

Underlying the principle that the fair value for rate-making purposes is the "actual cost of the identical plant", is the theory that the owner of the property has a right to a fair return on the money actually invested. Such a theory has much to commend it and is widely held. Among the theories of valuation for rate-making, there is at least one other for which strong arguments can be made.

The advantages of competition are real. In ordinary business it is only when there are no restrictions, when trade is free, and when competition is unrestricted that the community is best served, that prices are lowest, that the ingenuity of manufacturers is stimulated to design the best machines, and that the capitalization of the competing concerns is kept down to a healthy figure.

A method of valuation for rate-making, which would retain for the public the advantages of monopoly and, at the same time, obtain for them the advantages of competition, would be ideal from many standpoints. It would insure to the public the advantage of the most modern machinery and the most approved methods. It would also insure to the company the full increase in the value of its properties and plant caused by the increase in the size and prosperity of the community. This latter increase is in part due to the service of the public utility corporation and, therefore, the corporation is fully as much entitled to it as the householder to the increased value of his house or lot, or the storekeeper to the increased value of location due to the improved character of the neighborhood developed around his store.

In a word, the purpose of valuation should be to restore values to a competitive basis. The rules governing the management and operation of a public utility company should differ very little from those of any other well-managed business. The community gives the company a certain monopoly. In return it has a right to all the improvements in the art and to sound business management. The city should pay prices which permit of ample allowance for depreciation and development and are commensurate with those in other lines of business.

If in ordinary business a man makes a mistake and invests in a machine which is useless or which shortly becomes obsolete, the community suffers very little. It is true that a certain amount of capital has been lost, for which, eventually, the entire community must pay,

Mr. Dodd.

but, on the other hand, the community does not continue to pay interest on the investment, nor does it continue to pay high prices for goods produced by an obsolete process.

Competitors are always ready to take advantage of all improvements and to undersell those who have made unlucky investments. Improvements are eagerly hailed, because they actually represent a gain in the wealth of the entire community, in spite of the fact that such improvements mean a serious loss to some person or group of persons.

Speaking generally, therefore, the valuation of a public utility company should be the cost to reproduce a substitute plant, such a plant as a competitor might build, not a valuation of the identical plant.

With this general plan to guide, namely, that the valuation is desired of the plant which a competitor would build to give the same service, it becomes possible to determine not necessarily all the steps of the valuation, but, in general, what is desired.

The value of the machinery should be that of the latest improved kinds to give the same service. The cost of the real estate should be its present-day cost. The value of the buildings should be their value as if built according to present-day prices of labor and material. The value of underground pipes and conduits should be the value according to present-day prices of labor and material and present types of pavements.

On the other hand, the unit prices should not be those of to-day nor of yesterday, but the average prices for a number of years past.

As regards the question of when it is satisfactory to value the identical plant and when the valuation should be of the substitute plant, the same rule of liberality should be adopted as that recommended on page 14 of the report, in regard to unused property. There is no question that a modern steam turbine is far cheaper and more efficient than a slow-speed, reciprocating engine of the type in favor, say, 10 or 15 years ago. There is no question as to the comparative advantage of a modern water-gas producer over the old type of gas retort. There is usually no question regarding the comparative suitability of three 8-in. gas mains or a single 14-in. main. When there is no question that the machines actually installed are obsolete, inefficient, and expensive, the valuation should be of a substitute plant; however, when there is room for question, it should be that of the "identical plant."

With respect to the value of real estate and of buildings, there seems to be little difference between what is recommended in the report as regards the valuation of old properties and the method advocated by the speaker for the valuation of all properties.

As to paving over conduits and pipes, it is as much an increment to the value as the development of real estate near a railroad track, an addition to the cost to reproduce the road, and a just reason for in-

Mr. Dodd. creasing the taxes on the road. The paving adds to the cost if the competitor wishes to build; so also do the buildings restrain competitive building, in the case of a railroad. It is true that the company pays nothing for the cost of the paving, but, on the other hand, neither does the railroad pay for the buildings. Each case is a natural result of the growth of the community caused in part—sometimes in large part—by the company.

The full increase in the cost of the mains due to the paving should not be added to such cost, but only a proportion, depending on their age and estimated life. The cost of installing a competitive line is the same, regardless of the time when the paving was laid down, whether a year before or 20 years before. Therefore, if, for example, 80% of the estimated life of the main has expired at the date when the street was paved, only 20% of the added value due to the paving can be added to the capitalization of the company. In the same way, if the estimated life of the mains is 20 years, the annual depreciation which can be charged on the paving is the same percentage as on the mains, namely, 5% of the total additional cost to reproduce due to paving.

Although the report is notable for its clearness and for the many valuable contributions it has made to the science of rate-making, now in its formative stage, it appears to be open to criticism in one respect, namely, that it is difficult to determine the ground principles which underlie the proposed method of determining the cost to reproduce.

In general, it favors the actual cost of the identical plant. In the case of an old property (page 9), a deviation from this is recommended, because it is claimed to be impossible to determine the actual cost. It is difficult to accept the reason. There seems to be no insuperable difficulty in the way of determining the original cost of the labor and material of the identical units now in service.

For unused property, the report shows (page 13) that this rule is generally abandoned by the Courts, and that such property cannot be included, either at the original, or at any other cost.

As regards the present or original condition of the land (page 15), the Committee recommends that valuation should be based on the physical conditions existing at the time the property was built, but at present-day prices. In other words, it abandons wholly the idea of original cost.

When property has been donated (page 18), present-day prices should be used. Here, also, the principle is violated.

With regard to unit prices (page 30), the Committee recommends that present-day prices should be used for labor and material. The reason for this is not because of the difficulties attending the determination of original prices, but because Court decisions are in favor of present-day prices.

It appears, therefore, that the principle of permitting the company to earn on the actual original cost of the present property is one which the Courts have apparently refused to uphold; but, on the contrary, in all Court rulings to which reference is made in the report the present-day cost is the only one which has received their sanction (page 20). Mr. Dodd.

The Courts also generally refuse to sanction unused or abandoned property as being included in the assets (page 13). Is there good reason to doubt that they will consider obsolete inefficient apparatus in the same manner, and insist on a valuation of such machinery on a basis of the service rendered?

The Courts have granted the companies the right to the full increase in the value of the real estate, caused by the increase in the wealth of the community. They have granted the companies the same right to an increase in the value of buildings. It seems fair to assume that the Supreme Court will fail to grant a similar increase in the value of the mains as affected by the pavement.

It is believed that the method offered herein of valuing a plant on a competitive basis, that is, purely present-day prices of a substitute plant, would offer fewer inconsistences and would be more permanently satisfactory.

Morris Knowles, M. Am. Soc. C. E.—It is not to be presumed that the Committee's report, on a question still so much in process of development, can at any time, now or after much discussion, result in the complete agreement of the membership, or in the expression of any conclusive opinion by the Society. Members will gain, however, by the free interchange of opinion, will have their ideas clarified, and may be able to agree on some few fundamental principles. The correct application will continue for a long time to appear elusive. To aid in the discussion and understanding of the report, a tabulated list of definitions, or glossary, of the important terms used would be of great help, and it is to be hoped that the Committee will add such a list before concluding the report. Mr. Knowles.

The speaker's remarks relate largely to public utilities of restricted extent, with which he is more familiar, and not to railroads, because he can readily realize that the principles, at least for the present, that apply to such a utility, reaching to many different places, affecting many different peoples, and so many kinds of service, may well be different from those applying to a regulated monopoly in a narrow field.

Purpose of Valuation.—The Committee limits its report, by title and by explanation in the opening paragraphs, to the subject of "Valuation for the Purpose of Rate-Making", and states (page 5) that valuations for different purposes may not give identical results, "if the laws of the different States and certain decisions of the Courts are complied with".

Mr. Knowles.

In another paragraph (page 6), the Committee states:

"It is recognized, however, that the whole subject is in a developmental stage, and while your Committee has been guided largely by the decisions of the higher Courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions."

To the speaker there seems to be a certain degree of inconsistency in these statements. It is true that the subject is in a developmental stage and that the laws and decisions are constantly changing. It is common knowledge, also, that many of the recent decisions on these questions have been strongly influenced by consideration of expert testimony by engineers. May we not hope that a still larger influence in this direction will be enjoyed by this Society and its members? And, under the present conditions, would not the Committee accomplish its greatest usefulness by considering this, as it does other topics, from the point of view of sound principle, attempting to formulate true ideals, and by recommending practices which approximate these ideals as closely as existing laws and decisions permit? When authorities differ so widely, this would seem to be much more preferable than merely recognizing existing conditions and practices, and attempting to formulate ideals from them.

President Wilson has aptly stated "the day of accommodation, of concession, and of common understanding is, of necessity, a day of achievement". We are rapidly coming to the common understanding that fair return means equity, that it has all the force of its literal significance, alike to server and to user, *viz.*, an equality in value between the wealth, labor, and effort devoted by the owner to the public use, and the money returned to him by the public in rates. When this has finally been thoroughly established, a consideration of the matter in the light of principle will lead, the speaker believes, to the conclusion that valuations for all purposes, under complete and comprehensive regulation, should be identical.

It is true that no useful purpose would be served now by attempting to insist under the present laws that valuations for taxation be identical with other valuations, but the reason for this is that the problem of valuation for taxation is not a problem in valuation at all, but one in taxation. The method of valuation for taxation is fixed by the tax laws, and not by any consideration of the true meaning of valuation. The speaker believes that when an ideal system of taxation has been worked out, if it is based on property valuation at all, and requires valuation of public utilities for taxation, such valuations will be identical with those for purchase, capitalization, and rate-making. Until that time, the varying requirements of taxation laws cannot be adduced as throwing any light on the subject of valuation.

The following statement is made, on page 8 of the report: Mr. Knowles.

"As the future earning capacities of such properties under rate regulation by commissions will to a large extent depend upon rates to be established as a result of valuations of the properties, it is clearly illogical to base the valuations on present or past rates. Such valuation would be reasoning in a circle."

If such be the case, and if in the future the purchaser will be held down, and if he cannot earn on a different or greater value than that fixed by public regulation for rate-making, where will be the incentive to pay more for the property; what will keep alive the spirit of gambling in the future?

If it is agreed that a fair return is to be permitted on everything that the owner of a public utility has devoted to the public service, then the value for rate-making purposes will be the total actual reasonable investment, including all preliminary and development expenses and every sacrifice of time and labor, as well as of money. If returns on this value are limited to the market rate of borrowed capital under the given conditions of risk, then the value of the property for purchase would be that return capitalized at the same market rate of interest, or a value identical with that for rate-making purposes; and, if capitalization is to be controlled, so as to represent the actual investment. either of the original owner or of the purchaser, then the valuation for capitalization will be identical with those for rate-making and purchase.

The speaker then, while recognizing the laudable desire of the Committee to limit its report to one subject at a time, suggests that it is unwise to place on record, even by implication, that, either for present legal reasons or otherwise, valuations for different purposes will necessarily give different results. It would seem as if the Committee could be of more service by insisting throughout on the principles involved, instead of compromising, as in the statement first quoted above, with existing laws and decisions of Courts.

Present Conditions to Govern.—The report recommends: "the method of valuing the substantially identical property" according to "original conditions"; but goes on to say "valuations should be based upon the physical conditions at the time the various portions were built, but at the prices prevailing at or near the time of the valuation." This is manifestly an inconsistency, and an unnecessary straddle.

Let us analyze the attempts on the desire to do justice to the owner and rate-payer: It is generally recognized, as so ably stated by the Committee, that original cost, to include all proper items, and elements, and percentages, will give the desired result, if obtainable. It is rare that this can be determined, and this has led to the method—born of necessity—of estimating the cost-of-reproduction-new; determining

Mr. Knowles.

all factors as of the time of valuation, with normal prices for the preceding few years; but this should never be considered an end in itself, but only a means of throwing light on the probable original cost; and, if this is kept in mind, there will be no excuse for confusion arising from doing partly one thing and partly another. Granted that full historical and book cost may be difficult, even impossible of complete determination, and that, better than present-day-cost-of-reproduction is the cost-of-reproduction-with-original-conditions; the speaker submits that if "original conditions" can be determined, or even approximated, it is also true that prices at the time of construction can be even more accurately found, as these are frequently of record in the engineering and business periodicals, and in published reports and contracts of the time of construction. By using these we shall thus have harmony and consistency of thought and purpose.

Depreciation.—The Committee's statement on this subject is perhaps the most novel presentation in the report, and will arouse the greatest discussion. Time has not permitted an adequate analysis of the supplemental statement on this subject, but just a word may not be amiss. We have heard a great deal of the equity of charging rates upon 100% of cost new, on the basis that the service is kept up, that it is of full worth and therefore should be paid for accordingly, no matter what is done with the depreciation fund. This, we are told, is the concern of the stockholders. Now, much of the confusion and disappointment in the past have come from the endeavor to secure from these same stockholders the full repayment to keep up plant and service; and these stockholders are too frequently the "ubiquitous widows and orphans", who have acquired their stock from original owners who "got out from under" after drawing dividends at the expense of the depreciation fund.

Perhaps it would help us understand the position, so ably presented by John W. Alvord, M. Am. Soc. C. E.,* when he says the value for calculating rate of return depends on whether the fund has been kept intact, if we would remember not only that this fund, made up by annual payments by whatever system, is due the owners, that is, shall be paid in each year by the rate-payer, but also that the owner must preserve this fund intact for the purposes for which it was created. That is, it is a trust fund, and the owner has all the solemn duty of a trustee to preserve the fund for use in keeping up the property for the user who has paid into this fund.

It may even be true, in some cases of stagnant property and in order that refunding of bonds at maturity may be possible, that some small payment to a sinking fund as well may be necessary. This has been hinted by the Committee on page 7, when it says:

* *Transactions*, Am. Soc C. E., Vol. LXXVII, p. 788.

"It is not enough * * * that the borrower should pay interest on the money loaned, but he must also pay the principal to cancel his indebtedness." Mr. Knowles.

TABLE 19.—STATEMENT OF ENGINEERING COST ON CONSTRUCTION, BETWEEN APRIL 1ST, 1904, AND JANUARY 31ST, 1910, ON PITTSBURGH FILTRATION WORKS.

Contracts.	Final estimates.	Engineering Cost. Total.	Engineering Cost. Percentage of final estimates.
1. Slow sand filters, basins and appurtenances.....	$3 359 161.49	$332 854.65	9.91
2. Filtered water reservoir and appurtenances.....	415 245.28	46 398.07	11.18
3. River crossing and connections at Brilliant......	266 474.36	27 944.26	10.47
4. Centrifugal pumps, engines, generators and appurtenances, at Ross pumping station	92 956.30	4 385.76	4.72
5. Boilers, economizers, piping and appurtenances at Ross pumping station........................	68 403.53	2 676.59	3.94
6. Sand washers, electric machinery and auxiliary equipment for Ross pumping station..........	122 568.21	11 804.29	9.60
7. Ross pumping station............................	356 580.69	41 324.24	12.58
8. River wall, intakes and connections and grading, pumping station and machinery foundations..	208 318.92	26 933.76	12.94
9. Pumping engines, boilers and appurtenances at Brilliant pump station.........................	218 028.37	9 767.27	4.48
10. Pipe line and appurtenances, Highland Reservoir No. 2 to South Side..............................	555 250.76	34 877.71	6.28
	$5 662 987.91	$538 966.60	9.52

TABLE 20.—FINAL DISTRIBUTION OF CONTRACTOR'S COSTS BY BUREAU OF FILTRATION, ON PITTSBURGH FILTRATION WORKS.

	Contract No. 1. Slow Sand Filters and Appurtenances.		Contract No. 2. Filtered Water Reservoir and Appurtenances.		Contract No. 3. River Crossing and Connections at Brilliant.	
	Amount.	Percentage of total.	Amount.	Percentage of total.	Amount.	Percentage of total.
Materials and sub-contract items........	$1 733 378	47.2	$165 758	35.3	$105 567	35.0
General supplies..........................	127 402	3.4	16 087	3.4	10 327	3.4
Plant	275 249	7.5	34 766	7.4	22 318	7.4
Pay roll......................................	1 164 613	31.8	206 113	43.9	133 572	44.3
General expense............................	254 672	6.9	32 156	6.8	20 644	6.8
Interest and general overhead charge..	115 250	3.2	14 570	3.2	9 342	3.1
	$3 670 564	100.0	$469 450	100.0	$301 770	100.0

Engineering and General Expenses.—To those who have viewed with wonder the generally accepted basis for costs of engineering services of 5% and 6%, it is gratifying to note the results of the Commit-

Mr. Knowles. tee's inquiries into such costs on great and important water-works and transit systems, revealing amounts of 8 and 10 per cent. The speaker is glad to add one more set from the records of the Pittsburgh Filtration Works, from 1904 to 1910, inclusive, in Tables 19 and 20.

Extra Capacity.—The position of the Committee in regard to valuing all the property, including reasonable allowances for increased size, capacity, and availability for the future, cannot be too earnestly commended. No one with judicious foresight should be held down to designing works for the present only. Reservoirs, pipe sizes, filters, buildings, pumps, all need to be planned in varying degrees for the probable needs of a number of years. This is but ordinary good business judgment, and the sooner we can get the Courts to recognize this principle the better for all.

Mr. Leighton. M. O. Leighton, M. Am. Soc. C. E. (by letter).—The writer will participate in this interesting discussion only to the extent of calling attention to an interesting Court decision with reference to a much disputed point in the determination of "replacement value".

Mr. Allen Hazen, in his discussion of this report, makes the following statement:

"* * * in estimating the cost of reproduction of a water-works plant, the cost of cutting through the paving of the street over it, and replacing such pavement after laying the pipe, is unquestionably a part of the cost of reproduction, and must be included in any true estimate of it. The Committee, however, would exclude all that part of the cost which relates to paving laid after the pipe was placed. The writer is unable to see that the question of whether the pipe or the paving was laid first has anything to do with the cost of the reproduction of the property as it now stands, or with its value. It would cost just as much to replace the pipe in one case as in the other. The value of the property, as measured by its capacity for useful service, is identical in both cases."

On March 24th, 1914, the Court of Appeals of the State of New York, handed down an opinion in the case of the Kings County Lighting Company *v.* the Members constituting the Public Service Commission of the State of New York for the First District. Among the questions certified for decision were the following:

"Was the relator entitled, upon the facts shown in the record, to have the cost of reproduction of paving now in the streets but not in place at the time the mains were laid, allowed for in ascertaining the value of its property used in the public service?"

"Was the relator entitled, upon the facts shown in the record, to have the cost of reproduction of paving now in the streets but not in place at the time the mains were laid, allowed for in ascertaining the capital actually expended?"

With reference to the two foregoing questions, the Court said: Mr. Leighton.

"In determining the cost of reproduction, the Commission allowed $12 717 as the cost of restoring the pavement as it existed when the mains and service pipes were laid in the streets. The relator claimed an allowance of at least $200 000 for the cost of restoring pavements subsequently laid, on the theory that that cost would have to be incurred if the mains were to be laid to-day. But the new pavement in fact added nothing to the property of the relator. Its mains were as serviceable and intrinsically as valuable before as after the new pavements were laid. The controlling considerations under the preceding point also determine this. The rights of the public are not to be ignored. The question has a double aspect. What will be fair to the public as well as to the relator? (Smyth *v.* Ames, *supra*). Should the public pay more for gas simply because improved pavements have been laid at public expense? It is no answer to say that the new expensive pavements suggest improved conditions which, though adding to the value of the plant, will not, by reason of the greater consumption, add to the expense per thousand feet of the gas consumed. The public are entitled to the benefit of the improved conditions, if thereby the relator is enabled to supply gas at a less rate. The relator is entitled to a fair return on its investment, not on improvements made at public expense. It is said that the mains will have to be relaid. So will the new pavements, and much oftener. Both might possibly be relaid at the same time. The case is not at all parallel to the so-called unearned increment of land. That the company owns. It does not own the pavements, and the laying of them does not add to its investment or increase the cost to it of producing gas. The cost of reproduction less accrued depreciation rule seems to be the one generally employed in rate cases. But it is merely a rule of convenience, and must be applied with reason. On the one hand, it should not be so applied as to deprive the corporation of a fair return at all times on the reasonable, proper, and necessary investment made by it to serve the public, and, on the other hand, it should not be so applied as to give the corporation a return on improvements made at public expense which in no way increase the cost to it of performing that service."

J. P. Snow, M. Am. Soc. C. E.—The following has been suggested by the printed discussions of the report which have so far been issued. Mr. Snow.

In formulating its report, the Committee assumed certain fundamental principles to be axiomatic which several critics of the report have considered open to discussion. An explanation of a few of these will render the discussion clearer.

First.—It is claimed by some that valuations cannot be used in fixing rates for public utilities, especially for steam railroads. It seems idle to make this claim, in view of the facts that an Act of Congress provides for the valuation of railroads, and the Interstate Commerce Commission and its engineers state that valuations are an important factor in fixing just rates, and must be ascertained at whatever cost for their information and use. The railroads of the country are preparing

Mr. Snow. to make these valuations, and the Government engineers are organizing their forces to check the work. Where, then, is the force in the argument that these valuations cannot be used, and that the Committee should not have considered valuation for rate-making purposes in its report.

Second.—Several of those who have discussed the report claim that, because an item of property is rendering efficient service, although it is confessedly near the end of its life, it should be scheduled as having value new. This idea rests on the theory that rates should be based on the value of the service to the consumer, instead of on the cost of the service to the producer. Although aware that in the last analysis the law of supply and demand rests on a balance of these two theories, the Committee is firmly of the opinion that the value of service to the consumer cannot be used as the basis for computing rates in terms of money. Property must be kept in a state of full efficiency; else rates cannot be collected at all. When an item reaches the stage where its efficiency is practically impaired, it must be replaced and either scrapped or relegated to other parts of the service where less efficiency suffices. It follows that continued efficiency cannot be considered as a reason for holding an item in the schedule at value new throughout its term of life.

Third.—Protection of the investment seems to be another stumbling block with many who have discussed this question. The aim of the Committee has been to protect the owner in the legitimate, proper value of his property, based on present prices and original conditions so far as they have largely controlled the cost and are definitely known. If investment *per se* is to be considered, shall it be the original cost of an old property, or the cost to the present owners? Is the cost entirely legitimate? Has there been appreciation of value in some part of the holding? Has some part of the property been acquired by gift? These and many other questions render a full dependence on investment cost impossible and unjust—unjust sometimes to the consumer and sometimes to the owner. Justice seems to require that original conditions be considered as above, especially as regards the improvements on real estate, paving over pipes already laid, etc., because the legitimate, proper cost, so far as ascertainable with certainty, plus appreciation and minus depreciation, is the value. Higher present unit prices are an item of appreciation; and lower, of depreciation. Paving over pipes is a factor of added cost when replacement occurs, plainly chargeable at that time to capital as betterment, under the Interstate rules for railroad accounting.

Although the consideration of original conditions to any extent seems to hark back to that of investment, it does so only in so far as strict justice, to the owner on the one hand and to the rate-payer on the other, demands.

Mr. Snow.

It must not be supposed that hard and fast rules can be adopted for solving the problem of valuation for rate-making purposes, and can be extended to cover abnormal cases. Tolls on the Panama or New York Canals cannot be fixed by the Committee rules any more than can the price of diamonds from the Kimberly Mines. The application of any rules on the subject must be tempered by common sense, and this necessity has been kept in mind at all times by the Committee.

Referring to the discussion by Messrs. George F. Swain, D. W. Lum, and others: The word "rate", as used by the Committee, should be assumed to mean the gross income from the physical property which is valued; and does not go into the intricacies of a schedule of individual rates like a railroad tariff sheet.

The difference between using interest return on value new plus a sinking-fund allowance, as advocated by George F. Swain, Past-President, Am. Soc. C. E., and the Committee's recommendation of interest on depreciated value plus a depreciation allowance, depending on the life of an item and its age, lies more in the bookkeeping than in the amount of money involved.

Mr. Swain's is the easier method for the engineer, in that only the value new of the item and its term of life are needed, while the other method requires in addition the age of the item. The objections to his method are serious, however, and apparently very practical. The speaker begs leave to differ with him by thinking that the sinking-fund method is the ideal, impractical one and the Committee's the practical method. With the sinking fund the annual payments with their interest must be kept intact as funds which the stockholders must not profit from, else there will not be sufficient in the fund when the time for replacement comes. Now, in railroad practice, such funds can be used much more profitably in betterments to the property in general, where, of course, they will earn the higher rate assumed as possible by the rest of the property, than if loaned as ordinary investments at interest. This, then, would be the obvious and proper way in which to invest them; but, if so invested, the company must keep their earnings separate, except as used for straight replacements, rigorously withholding all earnings from betterments created by such funds from dividends, else the replacement fund will not be kept inviolate.

It is assumed that Mr. Swain would allow the return from value new as available for dividends, while the sinking-fund accumulation would provide at the proper time for replacements. If these annual sinking-fund allowances are used for additions and betterments, as is shown above to be advisable, can he separate these "fund items", as they might be called, so that the interest allowed on their value new would not be paid as dividends, instead of being held as they should be to build up the replacement fund?

Mr. Snow. Another objection is the repeated findings of the Supreme Court that an item cannot be held as of the same value as when new after it has passed an appreciable portion of its life term. If we may ignore the Courts, as has been suggested, many things not now thought advisable may be done; but, when these findings appear to be equitable and good law to so many of us, it is a question if it is not best to conform to them with good grace.

The Courts also set forth valid objections to a utility company accumulating a fund of any kind, but hold rather that it should pay as it goes, collecting sufficient revenue to do so.

By the replacement method, as advocated by Mr. Lum, the result is that the customers after the replacement have to pay the bill, rather than those who have enjoyed the use of the item while it was in service. This is seen more plainly in the case of a large item, like a water-works reservoir or a terminal railway station, than in the case of railroad ties, although the principle is theoretically the same.

To show that the method of the Committee in allowing return on remaining value is just and covers the exact new capital put into the property, Table 21 is submitted. Assume a business consisting of ten items of equal value, with life terms of 10 years, and that one item is installed each consecutive year until the ten are in use. The value new is represented by 100, and depreciation is computed by the equal-annual-payment method, recommended by the Committee, on a 7% basis.

The remaining value of each item at the end of each year is shown in the body of the table, and Column 12 shows that after one cycle, *i. e.*, 10 years, the total of the remaining values remains constant at 605.39. Column 14 shows that this sum also represents the total new capital that has been put into the property, assuming that the regular allowances for depreciation each year have been used toward establishing the new item due the following year. Interest on this sum (605.39) plus depreciation, as recommended by the Committee, is manifestly the proper measure of the return from these items of property to use when fixing rates.

If the straight-line method of depreciation had been used, the resulting total remaining value after a life cycle had passed would have been 550.00, and this figure would represent the amount of new capital needed under the conditions named above.

The reason for the smaller capital investment when depreciation allowances are computed by the straight-line method is, of course, that the early payments are large and consequently a greater return accrues to the owner's benefit than happens under the equal-annual-payment method. The final outcome at the end of the life cycle is the same if interest is fully allowed for in both cases.

Mr. Snow.

TABLE 21.—LIFE HISTORY OF A PROPERTY CONSISTING OF TEN EQUAL ITEMS WITH LIFE OF 10 YEARS. ASSUMING THAT ONE ITEM IS ADDED EACH CONSECUTIVE YEAR.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
	DEPRECIATED BY THE EQUAL-ANNUAL-PAYMENT METHOD. 7% RATE.												
Years.	Items of Property.										Value new, less depreciation.	Depreciation allowance.	New capital required.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.	No. 9.	No. 10.			
	Remaining value.												
0	100.00										100.00	....	100.00
1	92.76	100.00	....	....	....	....	....	....	....	....	192.76	7.24	92.76
2	85.02	92.76	100.00	....	....	....	....	....	....	....	277.78	14.98	85.02
3	76.73	85.02	92.76	100.00	....	....	....	....	....	....	354.51	23.27	76.73
4	67.86	76.73	85.02	92.76	100.00	....	....	....	....	....	422.37	32.14	67.86
5	58.38	67.86	76.73	85.02	92.76	100.00	....	....	....	....	480.75	41.62	58.38
6	48.23	58.38	67.86	76.73	85.02	92.76	100.00	....	....	....	528.98	51.77	48.23
7	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	....	....	566.34	62.64	37.36
8	25.74	37.26	48.23	58.38	67.86	76.73	85.02	92.76	100.00	....	592.08	74.26	25.74
9	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	100.00	605.39	86.69	13.31
Total new capital in plant........													605.39
10	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	92.76	605.39	100.00	0
11	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	85.02	605.39	100.00	0
12	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	76.73	605.39	100.00	0
13	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	67.86	605.39	100.00	0
14	67.86	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	58.38	605.39	100.00	0
15	58.38	67.86	76.73	85.02	92.76	100.00	13.31	25.74	37.36	48.23	605.39	100.00	0

Mr. Snow. To show the advantage of the equal-annual-payment method over the straight-line method when rates are to be determined, the diagram and table, Fig. 7, has been prepared. The depreciation by the straight line is, of course, 10 each year. This, added to the interest on the remaining value each year, is shown graphically at the left of the

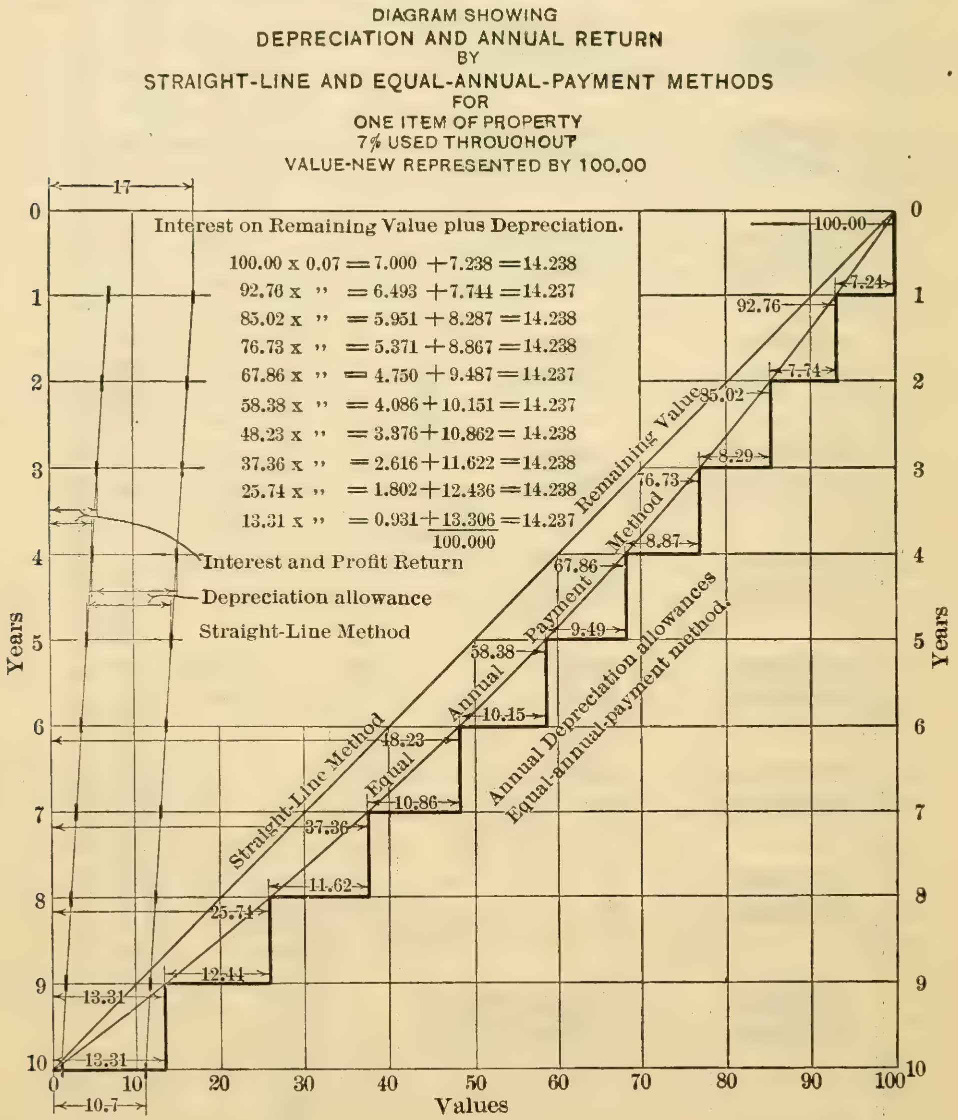

FIG. 7.

diagram. The total return will be seen to vary from 17 at the end of the first year to 10.7 at the end of the cycle, averaging 13.85. The similar features by the other method are shown by the table on Fig. 7, and will be seen to be constant for each year at a value of 14.24. It will be appreciated by all that level rates year by year are to be preferred to those which vary.

The interest and profit returns on remaining value, varying from 100 the first year to 605.39 at the tenth, by Column 12 of Table 21 is the proper amount to distribute as dividends; and the amounts in Column 13, of course, should be put back into the property. The figures of this column apply to the beginning of the year at which they are placed, or to the end of the preceding year, and, to obtain total return, must be added to the interest and profit return on the capital in the plant during the preceding year. If this is done it will be seen that the rates or gross income should be 14.238 at the end of the first year and direct multiples of this for other years up to the tenth; after which it remains constant at 142.377, or ten times the first return, as it should, as there are then ten items of property involved to one item at the beginning. The sum of Columns 13 and 14 at the beginning of the tenth year is 1 000, equalling ten full items, as it should. Mr. Snow.

If items of varying value or of irregular periods of installation had been assumed, the calculation would have been more complicated and the result not so clearly shown, but the principle would have been the same, and the equitable division between dividend and depreciation monies as clearly set forth.

If the sinking-fund method is used, the figures of Column 13 will be multiples of the first figure, provided the funds are invested outside, as a regular sinking fund should be. If put back into the property, the column becomes quite complicated, and the proper return available for dividends still more so. Under this last assumption, however, the actual amounts of new capital put in and the yearly returns will be the same as with the equal-annual-payment method shown, but, if the percentage of value new is distributed for dividends, the depreciation funds will suffer.

If, under the sinking-fund method, each item is paid for with new capital and the depreciation fund is invested outside, the new capital at the tenth year will be 1 000, the annual percentage on which is 70, and Column 13 will consist of multiples of 7.2377, which is its first term, the tenth being 72.377, as by the other method. This shows that the difference between these methods is one of bookkeeping, and the objection to the sinking-fund method is that a separate outside fund must be maintained or great risk run of distributing too much in dividends, thereby robbing the depreciation fund.

Considering the replacement method advocated by Mr. Lum, Column 13 would consist of zeros to the end of the tenth year. From the eleventh year onward the figures would be 100. The new capital would increase by 100 each year for 10 years, and the annual returns would be 7 for the first year and multiples thereafter to the tenth year. Hence the public should pay for the ninth year 63, for the tenth year 70, and for the eleventh year 170. This would necessitate

Mr. Snow. a great jump in the rates or a cut in the dividends. For ordinary railroad operation, renewals can be arranged so as to be fairly uniform from year to year, but when a great terminal is rebuilt, the above jolt occurs and, as it is difficult to raise rates, dividends must suffer, as local instances show.

Careful study of the tables in the report and the various ones since submitted by the Committee will show the entire equity of the equal-annual-payment method and its certainty to protect the company owning the property considered.

Mr. Mills. ARTHUR L. MILLS, M. AM. SOC. C. E. (by letter).—When the real estate of a company is of comparatively small value, the treatment of appreciation is of no great importance, as earnings cannot be provided by regulation with any degree of precision; but when the value is large, it may become a serious problem.

The value of the real estate of the four principal railroad lines to Chicago, the New York Central, Pennsylvania, Erie, and Baltimore and Ohio, probably exceeds $1 500 000 000. The appreciation of this real estate has averaged at least 5% per year in the last 20 years. If this appreciation is, in the future, to be considered as part of the income, it means that the estimated income will be greater than the actual income by $75 000 000 per year; whereas the estimated increase in income for all the roads in this territory by the 5% advance in rates which these roads desire, is only about $40 000 000.

The U. S. Supreme Court has decided that the cost of reproduction or present value of the property must include the value of the real estate at the time of the valuation.

The New York Public Service Commission in the First District includes in the probable income the estimated appreciation of the real estate; but the Appellate Division of the Supreme Court of the State of New York, in a decision rendered in May, 1913, refused to adopt this view. The inclination of the Interstate Commerce Commission at that time is indicated in the decision in the Western Rate Case. Commissioner Lane said on this subject:

"Whatever the true economic or legal view may be as to the right of a carrier to consider the increase in value of its land as a part of the value upon which it is entitled to a reasonable return, such increase in value does not of itself establish the right of a carrier to increase rates upon a given service. Certainly if the Supreme Court may decline to lay down the absolute rule that 'in every case failure to produce some profit to those who have invested their money in the building of a road is conclusive that the tariff is unjust and unreasonable', *Regan v. Farmers' Loan & Trust Co.*, 154 U. S., 412, it is a conservative statement of the law to hold that a railroad may not increase the rates upon a number of commodities solely because its real estate has arisen in value."

"While it is evident, therefore, that each case must be decided upon the facts peculiar to it, the Commission believes it proper in this case to follow the general rule, as stated by Judge Hough of the United States Circuit Court (Consolidated Gas Co. *v.* City of New York, 157 Fed. Rep., 855): Mr. Mills.

" 'Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the one at bar.'

"Thus, land has been taken at its fair value and not at its original cost, and the annual appreciation of land has been treated as a profit. By this method, all property is treated absolutely alike, as Judge Hough suggests. No difference is made, except that as depreciation represents a decrease in assets, it is placed as a *debit* against operation, while appreciation is placed as a *credit,* because it is an increase in assets. Land has sometimes been treated like other property only to a degree; that is, each class has been appraised at its present worth or value. That has been done in this case. But if property has been taken at its *depreciated* value where it has depreciated, an entry must regularly be made in estimated operating expenses equal to the average annual depreciation. Conversely, if land, or any other property which genuinely appreciates in value, is to be taken at its *appreciated* value, then an entry must be made in the estimated receipts equal to the average annual appreciation. Unless this is done, it is obvious that the consumer will be burdened with all the estimated decreases in assets but not credited with the increases in assets. If the principle laid down by the court is to be followed in part, it should be followed in whole.

"It is suggested that the annual increase in the value of land which is treated as income is not actually received. Increase in the value of unoccupied land is not realized until sold or put to use, but it is real, nevertheless, although payment may be deferred. Likewise, payments to the depreciation fund are not actually expended; yet they have been considered legitimate charges in practically every case. Furthermore, the *annual* increment is no more indefinite than the *total* increment—the present value. But if the present value can be determined, it is possible to determine past *annual* appreciation with positive accuracy, for it is only a simple mathematical calculation. It is also probably as easy to estimate increases in the near future as it is to estimate what obsolescence, which is a form of depreciation, there will be in the future.

"Indeed, the problem of handling appreciation is much simpler than depreciation. If the property is growing more valuable, the investor need not worry; and if the state recognizes his right to earn a fair return upon the increase, he is fully protected. It is not necessary that the increase be represented by stocks or bonds, for if the earning power is there, he will receive a return thereon, regardless of the amount of securities. In fact, the existence of an increase which is not represented by securities is an element of safety, a reserve fund of a valuable kind.

Mr. Mills. "There is a further similarity. The exact amount of depreciation and the annual rate are not definitely known until the piece of property is actually replaced or has become useless. The total appreciation and the average annual rate are not known until the land is sold, but when it has been disposed of (and plants are continually being removed and the land sold), they become absolute certainties. Why should these matters be considered less definite when applied to land than when applied to the buildings thereon? The depreciation of the buildings is a charge against operation; why should not the appreciation of land be a credit?"

Mr. Lane's statement, "it is a conservative statement of the law to hold that a railroad may not increase the rates on certain commodities solely because its real estate has risen in value", may be fair as an abstract proposition; but it does not follow that an advance in rates should be denied because it would permit a return on the appreciated value of the real estate. The U. S. Supreme Court in many cases has laid down the general principle that the owner is entitled to fair return on the fair value of his property, regardless of cost, provided the rates are reasonable. It would probably be considered against public policy to allow railroads to raise their rates simply for the purpose of paying excessive dividends, even if such an income would be no more than a fair return on the value of their property. It does not follow that the cost of reproduction or present value will necessarily be the controlling factor in rate regulation. The Supreme Court has decided that the original cost, cost of improvements, and the value of the stocks and bonds, must also be considered, and that, regardless of everything, the rates must be reasonable.

Where a railroad needs more money to meet its expenses and also the demands of an increasing tonnage, and to keep up its dividend rate which is not excessive, however, it appears equitable to permit an increase in rates, provided the proposed rates are reasonable and the anticipated income is not more than a fair return on the fair value of the property, regardless of the original cost of the real estate.

Appreciation and depreciation are separate and distinct processes, and each should be considered on its own merits independently of the other and by reason of the effect it has on the present value. It is found necessary to provide from the earnings the cost of the depreciation of the structure, due to wear and tear, etc., for the reason that it is a necessary expense and one that cannot be neglected without damage to the property; but this in itself is not a valid reason for considering appreciation as an income.

Appreciation is not an actual receipt, and is of no assistance to the railroad in meeting its disbursements. To the extent, therefore, that estimated receipts include estimated appreciation, they will, for all practical purposes, be fictitious. Moreover, because real estate has appreciated at a certain rate in a term of years, it does not follow that

the appreciation will continue at that or any other rate. The value of real estate frequently remains unchanged for a long time and then increases rapidly for a few years. Any estimate of the future appreciation of real estate is speculative, as many purchasers of that class of property know to their sorrow. Mr. Mills.

Appreciation of real estate is available as income only when the land is sold, and, as a railroad is rarely able to sell its real estate, the stockholders cannot ever expect to realize any income from real estate appreciation.

The real estate of a railroad is different from that of most other corporations. In rare cases only is a road able to dispose of even a part of its land. The demands of a constantly increasing business make it necessary to purchase additional land. It is doubtful if the actual value of the real estate is given any consideration by investors in railway securities. To suppose that the real estate of a railroad may be sold, is to pre-suppose its bankruptcy and abandonment.

Appreciation of real estate differs in character from the depreciation of the physical property due to wear and tear, obsolescence, etc.; it resembles depreciation due to a decline in the prices of rails, equipment, etc., from the prices paid at the time of their purchase, but this kind of depreciation is neglected in a valuation.

Mr. Lane says that unless appreciation is considered a part of the income "it is obvious that the consumer will be burdened with all the estimated decrease in assets but not credited with the increases in assets". The consumer is not burdened with depreciation because it is a decrease in the assets, but because it is a part of the expense of operation. It is something that requires money, and unless appreciation can provide money it does not seem equitable to use it as an offset to depreciation or for any other reason.

One is almost forced to the conclusion that the idea of including appreciation as a part of income did not originate from considerations of equity, but from a desire to find some way to offset the effect of the continuing appreciation of real estate which, in time, will increase the physical values of railroads so that a fair return thereon will permit excessive dividends, and to prevent future reductions of the rates.

It does not seem logical, however, to expect that rates will remain stationary while the cost of handling the traffic is continually increasing. As the volume of traffic increases, it may offset in part the greater cost of labor, supplies, etc., but only in part, because increasing tonnage brings with it the demand for additional facilities.

If the appreciation of real estate is to be considered as a part of the income to which they are entitled, the railroads would be better off if their property did not appreciate in value. For, by considering appreciation as income, they would have to pay from their earnings the

Mr. Mills. full amount of this appreciation with the expectation that at some future time they would receive the interest thereon; but there is no certainty that they will receive this interest, for, at the time of the next adjustment of rates, the physical valuation might be neglected, other considerations being the controlling factors.

If part of the return to which a road is entitled is represented by the non-tangible appreciation of the real estate, there will be, theoretically at least, a deficit in money with which to meet disbursements. It has been suggested that securities can be issued to make up this deficit; but this, in effect, would be to pay operating expenses or dividends in part from the sale of stocks and bonds. It would hardly be equitable to ask a stockholder to waive his dividends wholly or in part because the real estate of the company was increasing in value, knowing that probably it will never be of any benefit to him in the matter of dividends or in enhancing the value of his stock.

The appreciation of real estate is no more a part of the fair return to which a railroad is entitled than losses incident to the enterprise are a part of the expenses on which rates are to be based. If the consumer does not guarantee losses, he is not entitled to the profits. Suppose it was found that the real estate of a railroad was depreciating, would it be considered fair to call this depreciation a part of the expenses to be provided for from earnings? Such a proposal would be decided negatively, for the reason that changes in value are a risk in his enterprise which the investor assumed.

Again, suppose there are two roads, similar in value, except that one owns its real estate and the other leases it at a rental based on its value, provision being made for revaluation. Then, if appreciation is considered as income, the second road will theoretically be allowed the larger rate of income from earnings, for the reason that the expenses of the second road will include the rentals, while the income the first road might be allowed on the value of its real estate will be offset by the appreciation.

As an illustration, take two railroads, one valued at \$100 000 000, of which \$30 000 000 represents the value of the real estate which has appreciated at the rate of 5% per year. The other road owns no real estate, and is valued at \$70 000 000. It uses real estate valued at \$30 000 000, paying therefor as rental 6% on its value. It is assumed that each company will be allowed to earn 6% on its valuation. The first road will be allowed to earn \$6 000 000, but in this income will be included \$1 500 000, which has been the yearly appreciation on its real estate, leaving a balance of cash available for interest and dividends of \$4 500 000, or 4½% on its value. The second road will include the rentals in its expenses and will be allowed to earn 6% on its value.

Theoretically, then, if the appreciation of real estate is considered as income, the road which owns its real estate is at a disadvantage as

compared with one that has no such investment, but pays a rental for its use. The word, theoretically, is used because regulation of railroad earnings with any degree of precision by changes in rates is impracticable. Railroad rate regulation is a more complicated problem than in the case of public service corporations with no competition and with service limited to a comparatively small area. Mr. Mills.

To consider the appreciation of real estate as income is confiscatory in character, for the reason that it deprives the owner from any benefit from the future appreciation of the real estate. If he pays for it yearly out of the "fair return" on the chance of getting interest on it some time in the future, the appreciation of his real estate has been practically confiscated.

Briefly, then, for the following reasons, appreciation of real estate should not be considered as income:

It is indeterminate and speculative;
It is not a tangible receipt;
It is different in character from the depreciation of the physical structure, due to wear and tear, for it is a matter of price which may advance or decline.

Such a practice would have the effect of giving the consumer the benefit of the profits due to appreciation, without requiring him to contribute to the losses of a similar character.

It would create a deficit, theoretically at least, which could be overcome only by the sale of stocks or bonds, which, in effect, would be to pay expenses or dividends in part from the sale of securities.

It is confiscatory, because whenever a property is sold the owner is entitled to its full value, whereas if he is required to pay for the appreciation of his real estate, he is deprived of a part thereof.

J. M. Schreiber, M. Am. Soc. C. E.—The speaker is particularly interested in the section of the Committee's report relating to depreciation, because he has been identified with similar work undertaken by the American Electric Railway Association. About three years ago, that Association, appreciating the importance of depreciation, through its allied associations, the American Electric Railway Engineering Association and the American Electric Railway Accountants Association, appointed a joint committee of six members, comprising three accountants and three engineers, to investigate the subject. This Committee is still in existence, and is known as the "Joint Committee on Life of Railway Physical Property". The subject was taken up with serious thought and energy, and sincere and earnest endeavors were made, in order to comply with its purpose. The data, secured from the properties with which the individual members were connected, were not only examined, but the Committee got in touch with qualified representatives of almost all the large electric railway companies in the Mr. Schreiber.

Mr. Schreiber.

United States. Amplified inquiry forms were sent out and personal solicitations were made, in an attempt to obtain figures expressing the life of railway physical property, or definite tables which would accomplish the same results.

In general, it was found that those having practical experience, and, in many instances, the responsibility of a particular item of railway property under investigation, felt that any attempt at present to express the life of railway property explicitly, was at best a guess.

After three years of study of the life of railway physical property, from every source obtainable, including the accumulation of a more or less complete bibliography on the subject, this Joint Committee reached the following conclusions:

(1) That the basic elements which determine the life of railway physical property which affect its usefulness are:

(*a*) Use,
(*b*) Climatic and soil conditions,
(*c*) Maintenance,
(*d*) Inadequacy,
(*e*) Obsolescence,
(*f*) The human element,
(*g*) The public demand, and
(*h*) Earnings.

(2) That it is not practical, or even theoretically possible, to assemble compositely these several elements into any form of logical "Life Table" that will apply equally in Maine and in California, in Minnesota and in Louisiana, or on different routes or lines of the same system embracing such physical property.

(3) That the ultimate solution of depreciation of railway physical property is insured earnings.*

It has happened that a great many of the points covered in the conclusions cited have practically been acquiesed in by those who have discussed the subject. It seems that the only way to consider depreciation, if at all, is as suggested by Professor Swain—to designate or set apart a certain portion of the earnings each year to be used for that purpose. Depreciation is an operating problem, and is simply deferred maintenance, and generally, if the amount is proper, allowing for reasonable contingencies, it is used up as operation proceeds year by year. This is because the railway physical property does not have to be replaced all at one time, but on a piecemeal basis, as it were.

Now, as to the important subject of proper general methods for valuation for the purpose of rate-making, it is the speaker's firm opinion that there is only one consistent method to follow, and that is to

* A full report of this Committee, with discussions, may be found in the *Proceedings*, Am. Electric Ry. Assoc., for 1912 and 1913.

ascertain the cost of reproduction new; and, that depreciation should not be taken into account. All the user is concerned about is proper service, and he does not care a whit about depreciation, if the service is not impaired. The speaker desires to call attention to a recent decision of one of the commissions, which is pertinent in connection with the report of the Special Committee on Valuation. It is contained in the Sixth Annual Report of the Public Service Commission of Montana for 1912 and 1913, in the case of J. F. Edwards *vs.* The Helena Light and Railway Company, in which the Commission says: Mr. Schreiber.

"Assuming that rates were being made for a new plant, it would be the total capital that must be considered as entitled to bear interest, as there would be no accrued depreciation. Depreciation is a liability against the property, which must be accounted for, but in making allowance for future depreciation, it is not the intention to provide for accrued depreciation, which it is assumed, has been taken care of. To simplify the matter, let us assume that an investment is made in 1903 of $100,000 under a twenty-year franchise, rate of interest allowable 10% per annum, and figuring 5% per annum depreciation. At the end of ten years, or in 1913, the property will have depreciated $50,000 and has a remaining value of a like amount. Then, if rates are made, based on the depreciated value, they must be one-half of the original rates, although the service may be just as efficient as it ever was, and in ten years more, the physical value of the plant would be nil, and likewise upon the same basis of reasoning, the utility would not be permitted to charge anything."

In conclusion, the speaker appreciates thoroughly the great task undertaken by the Committee on Valuation for the Purpose of Rate-Making, and thinks it deserves a great deal of credit. He agrees with a considerable portion of the report. As a member of the committee of the American Electric Railway Association, he is quite sure that it will be very glad to co-operate in every way with the Valuation Committee, and furnish any information and data on the particular subject of depreciation at its disposal. The speaker would also suggest that the Committee confer with the National Electric Light Association, which no doubt would also be willing to furnish pertinent data on the subject.

Certainly any conclusions on the whole report should not be accepted until the utilities, which are so vitally interested in the question, could at least have an opportunity to present their views properly.

J. P. Newell,* M. Am. Soc. C. E. (by letter).—Should depreciation be taken into account in fixing rates? If by this is meant, "Should rates be reduced as the depreciation increases?", the question must be answered in the negative. Assuming that the service rendered remains the same in quantity and quality, the rates should remain the same. Mr. Newell.

* This discussion was presented before the Portland, Oregon, Association of Members of the American Society of Civil Engineers, at its meeting of April 4th, 1914.

Mr. Newell. Any other policy certainly would not be attractive to capital, nor would it be of advantage to the public. As the total return during the life of the plant must be the same in any case, diminishing rates would require a higher charge in the beginning. Still, depreciation must be taken into account for the benefit of the company, not of the public, for depreciation is not a penalty assessed against the company for the deterioration of its plant; it is an amount added to the rates to repay the company for this same deterioration.

Much of the confusion of thought concerning depreciation arises from failure to distinguish between the standpoint of the public and that of the management. The prudent manager of a public service corporation will open a depreciation account, and will take good care that the amounts credited to it do not pass to the stockholders in dividends, leaving him unable to provide for renewals and replacements when needed. That, however, is of no interest whatever to the public. Fix rates that will insure capital a fair return, and renewals are certain to be taken care of. Even if the existing company dissipates its capital and is unable to replace it, some new company, knowing that a fair profit is assured, will be found ready to step in with new capital and take up the work.

The rate-regulating power need concern itself with only one question: At what figure must rates be fixed so that, at the end of the life of the plant, the owners will have received the full amount of the operating expenses, the value of the plant, and a fair return on their investment?

One method of finding the answer to this question is that used in dealing with a peddler: Find out the least he will take and give it to him. If we can do this, we need only divide the total amount by the assumed life of the plant, and we have at once the annual charge for service. The capitalist may put the receipts in his pocket, divide them up in his accounts in any way which suits him, and spend them as he pleases; and it is in accordance with this principle that the fairness of all rates must ultimately be judged. The least that capital will take, under conditions such that all capitalists are free to invest, is what it should be allowed to charge.

It would take years, in many cases decades, however, to apply this method. Besides, we must deal with many utilities, built under different conditions, the owners of which are not free to act on the above principle. We must deal justly with these men, who are, in a sense, at our mercy, or later we shall find ourselves at the mercy of other capitalists who will refuse to invest in other utilities which we need.

Some contend that rates should be based on the reproduction cost new and not on the depreciated value. True, as the rates must be sufficient to return to the owner an amount equal to this cost. The owner's profit, however, cannot justly be based on this reproduction

cost new, if part of it is returned to him annually through the depreciation allowance. Mr. Newell.

Compare the case with that of a loan. Suppose B borrows $100 from A at 6% interest, the principal to be repaid in 10 years. Anticipating B's inability to pay the whole amount at one time, A insists that B, in addition to the interest, shall make an annual payment on the principal. At the end of the first year B will then pay $10 plus interest on $100, or $16. At the end of the second year his payment will be $10, plus interest on $90, and so on to the last year when he will pay the last $10 of the principal and interest on $10.

Certainly no one will contend that A should continue to receive interest on the original $100 for the entire time, on the ground that he expects to make B a new loan at the expiration of that time. Neither should an investor expect to receive a profit on capital which has been repaid to him. Nor does this depend on the "adequacy" of the depreciation allowance. If that allowance be small, only a small part of the investment is wiped out, if large, there is a larger reduction in the capital; but the principle is the same in either case. If at any time it is found that the life of the plant has been assumed at too high a figure, and the rate of depreciation is correspondingly too low, the remedy is to increase the depreciation allowance, and, consequently, the rates charged the public, and to make this increase sufficient to cover the error for past as well as for future years. Care should be taken to assume depreciation high enough in the first place so that adjustments will reduce rather than increase rates. In fixing the rates of an existing plant, if it can be shown that its income has been too small in the past to produce a fair return, or that the profits have been excessive, the rates should be governed accordingly. If neither of these conditions can be satisfactorily proved, however, we must proceed on the assumption that the plant has been earning a fair return and no more.

Consider the case of a plant fully equipped, with business neither increasing nor decreasing, so that no extensions are required; the cost of reproduction new and the expected life of the plant have been ascertained, and the fairness of the existing rate is to be determined. The rate under consideration produces a gross income which is constant from year to year. The operating expenses are assumed to be constant, so the sum of the depreciation and the profit, or return on capital, the other two elements composing the gross income, is also constant.

If we adopt the "Straight-Line" Method of calculating depreciation, both these factors become constants, also; that is, we have a constant return based on a constantly diminishing capital. There is no inherent injustice in this to either party, but the rate of return on the existing investment must be too low at first, and too high at the last. This is illogical and unnecessary. It opens the way for complaint from the

Mr. Newell. company in the first period and from the public in the last. It adds also to the difficulty of adjustment, if the assumed rate of depreciation is found to be erroneous.

Let us then assume that the rate of return on the capital remaining in the plant must be the same each year. We now have these conditions: The gross income, a constant, is made up of three parts:

(1) Operating expenses, also a constant;
(2) Depreciation;
(3) Profit.

The sum of (2) and (3) is constant. The rate of profit or return is constant, but, being based on a valuation which diminishes year by year, the actual return also diminishes, and the depreciation correspondingly increases. The value and the life of the plant being known and the sum of the depreciation and the profit being fixed, as stated, it becomes a matter of mathematical calculation only, to assign the proper amount to each. The depreciation must be fixed at such an amount, for the first year, that the depreciation for each year plus the return on capital, based on the depreciated value for that year, shall equal the given sum, and that the sum of all the annual depreciations during the life of the plant shall equal the value of the plant. Solving, we get the rate of return on capital produced by the given rate, and the amount of capital repaid each year.

In the illustration already given of a loan, B pays $16 the first year and $10.60 the last. If, instead, he makes a payment of $13.59 each year, from which the interest due is first deducted, and the balance applied on the principal, he will likewise extinguish the debt in the given time. In the first year his payment on interest will be $6 and on principal $7.59; in the tenth year, the interest will be $0.77 and the principal, $12.82. Similarly, the annual repayment of capital from a uniform rate should increase as the depreciated capital diminishes. There should be no attempt to make the depreciated allowance conform to the actual visible depreciation from year to year. Fluctuations in the observed conditions should be taken account of only as they affect the life of the plant.

The results obtained by this method correspond to those given by the tables accompanying the report of the Special Committee. The Committee, unfortunately, has confused the matter somewhat by presenting a table in which one percentage is used in calculating depreciation, and another for profit. The sum of the results thus obtained not being constant, it follows that rates cannot be constant.

A concrete example will make these principles clearer. Assume a plant costing $100 000, with a life of 10 years, and operating expenses of $6 000 per year, and fix the fair rate of return on capital for this

kind of investment at 7 per cent. This gives all the information needed for the establishment of rates. Mr. Newell.

By the "Full-Value" Method, the annual depreciation will be \$10 000, the return on capital \$7 000, and the rates \$23 000.

The totals for the life of the plant will be:

Operating expenses........	\$60 000
Depreciation	100 000
Profits	70 000
Total	\$230 000

It will be seen from Tables 22 and 23 that the depreciated value of the plant at any given time has no bearing on the rates. The whole matter of depreciation is considered, once for all, when the length of time is determined during which the value of the plant must be repaid out of the rates. Any further evidence of depreciation is important only as indicating error in fixing that time.

In all the previous discussion, the effect of renewals and replacements has been ignored; but these do not affect the principles stated. The item replaced has been depreciated to zero condition, and then renewed to 100% by the introduction of new capital. The life of the plant is a composite of the lives of all the parts, in proportion to their respective values.

Fair rates may also be determined by the Sinking-Fund Method, provided the profit is based on the reproduction cost new, and the interest on the sinking fund is calculated at the same rate as the profit. Under this method, instead of handing to the owner each year certain varying amounts, the sum of which will equal the value of the plant within its life, and which are to be deducted each year from the capital on which a profit is permitted to be earned, a uniform amount, less than the average of those above, is assumed to be set aside each year. These uniform amounts, at compound interest, will equal the value of the plant at the end of its life, but they will not do so unless the owner continues to apply his industry and skill to them. As the capital is gradually transferred from plant to sinking fund, but remains always engaged, the fair profit must be based on the full, and not on the depreciated, capital. This is true, no matter by what means the owner chooses to make the sinking fund earn the required interest.

It is commonly assumed that interest on a sinking fund should be calculated at a very low rate, such as money can earn with no risk and with the minimum of attention. To the writer this appears clearly erroneous. The owner is paid not only for the use of his money, but also for his skill, industry, intelligence, and risk. These factors he is paid for applying to the entire capital, whether it be all in plant,

Mr. Newell.

TABLE 22.—BY "STRAIGHT-LINE DEPRECIATION."

Year.	1	2	3	4	5	6	7	8	9	10
Operating expense........	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000
Depreciation	10 000	10 000	10 000	10 000	10 000	10 000	10 000	10 000	10 000	10 000
Profit.....................	4 238	4 238	4 238	4 238	4 238	4 238	4 238	4 238	4 238	4 238
Depreciation value........	100 000	90 000	80 000	70 000	60 000	50 000	40 000	30 000	20 000	10 000
Rate of profit............	4.24%	4.72%	5.30%	6.05%	7.06%	8.47%	10.60%	14.13%	21.19%	42.38%

Annual rates..	$20 238	
Total operating expense..		$60 000
" depreciation..		100 000
" profits..		42 380
		$202 380

TABLE 23.—BY "EQUAL-ANNUAL-PAYMENTS."

Year.	1	2	3	4	5	6	7	8	9	10
Operating expense........	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000	$6 000
Depreciation..............	7 238	7 744	8 287	8 867	9 487	10 151	10 862	11 622	12 436	13 306
Profit.....................	7 000	6 494	5 951	5 371	4 751	4 087	3 376	2 616	1 802	932
Depreciation value........	100 000	92 762	85 018	76 731	67 865	58 378	48 226	37 364	25 742	13 306
Rate of profit............	7%	7%	7%	7%	7%	7%	7%	7%	7%	7%

Annual rates..	$20 238	
Total operating expenses..		$60 000
" depreciation..		100 000
" profits..		42 380
		$202 380

or partly in sinking fund. As he is paid, through the rates, for such management as will cause the sinking fund to earn the given rate, he cannot well claim that it is actually earning less. Mr. Newell.

The best possible proof of the correctness of these two methods of calculating fair rates is to be found in the fact that, though differing widely in principle, they give identical results.

If the principles governing depreciation have been correctly stated, it follows that as far as rate-making is concerned, the public service corporation need concern itself about "depreciated condition" not at all, but about the rate of depreciation a great deal. Proof of a short life for the plant will do more than anything else to justify high rates. Such proof will likewise establish a low depreciated value, which may affect the company's interests in the way of sale or taxation values, or borrowing power, but not in respect to rates.

The appraiser, instead of constructing mortality tables by which to determine the condition percentage, should observe conditions in order to make up mortality tables. Every item of the appraisal should have given its age and expected life, as well as the reproduction cost. Depreciation can be expressed only by the ratio of remaining to total life, as depreciated value is a function of the rate of return on capital as well as of length of life. The composite length of life of the plant must then be determined by calculation, the operating expenses by the company's accounts, and the fair rate of return by the judgment of the public service commission. From these three factors and the reproduction cost new, rates properly to be charged the public can be calculated by either of the methods discussed.

The writer prefers the Equal-Annual-Payment Method, because it takes account of all the facts all the time. It gives a "present value" which is also the true value for purposes of sale and taxation. As it reckons profit on depreciated capital, it is more nearly in accordance with Court decisions, and is more likely to meet with popular approval.

Gardner S. Williams, M. Am. Soc. C. E.—On its appointment, this Committee was designated a "Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities." The speaker regrets that thus far the Committee does not seem to have risen to the plane required for such a consideration of the problem submitted, as its importance demands, and as some, at least, of those who stood sponsor for the appointment had reason to anticipate. Mr. Williams.

In the speaker's understanding, the Committee was "to formulate principles," not to accept rulings already formulated under the exigencies of particular conditions, especially as in the majority of cases those rulings have been formulated by persons far less qualified to

Mr. Williams. deal with the subject, in its entirety, than it is proper to assume the membership of this Committee to be.

Further, it is the belief of the speaker that neither the Committee, nor any one else, should accept dicta as decision, nor opinion as law; or even accept decision as final. For 300 years the Courts steadfastly refused to admit opinion evidence, and it is within the memory of some of the members of this Society when it was absolutely excluded; but where in America to-day is the higher Court that does not receive and consider it?

To this Committee, in the discharge of its prescribed duty, as the speaker understands it, the existing interpretations of the law and the wording of special acts are of no consequence, until the Committee, after careful and mature deliberation, is prepared to say that those interpretations and the wording of those acts are just, equitable, and fair to all concerned in all cases. If the work of the Committee were to have been bounded and controlled by legal decisions and legislative enactments, it should have been empowered to employ those expert in the law to perform its work, or the whole question should have been referred to a bar association. If, in dealing with this subject, the American Society of Civil Engineers cannot get back to, and enunciate, those fundamental principles of rights and justice that underlie and overrule all man-made law, then, in the speaker's opinion, again, this Society should let this subject alone.

In importance, the question referred to this Committee overshadows every other question this Profession is now or has ever been called on to consider. Of the estimated fixed wealth of the United States, exclusive of real estate, nearly 60% was invested, in 1904, in properties within the purview of this Committee, and the investment therein has increased 50% in the last 10 years. On all hands the rumblings of a coming strife between the masses and the classes are to be heard. State socialism is being advanced as a panacea for the ills bred in ignorance and nurtured in indifference, and engineers, trained by the responsibilities of their profession, are the one class fitted to rise between the contending forces and present a correct and safe plan of future procedure. To no class other than themselves does the advance of socialism presage such disaster. Socialism stands for the mediocre man, for the ignoring of special ability, and the suppression of more than average energy. This Profession exists as the foremost exponent of special ability and increased efficiency. The two ideas are absolutely inconsistent and incompatible, and the engineer must recognize the fact and enter the combat accordingly. He must qualify himself to explain clearly to his fellow-citizens the true relations between capital and labor, and the rights and privileges of each, and must take the position, in the community, as a leader

of its thought, for which his training in exact reasoning should have prepared him. Mr. Williams.

The valuation of the property of public utilities may be undertaken for any one of five different purposes, which, in the order of their justification, may be stated as follows:

A.—For the purpose of forced acquisition;
B.—For the purpose of sale or purchase;
C.—For the purpose of taxation;
D.—For the purpose of issuing or limiting the issuance of securities;
E.—For the purpose of establishing rates.

(*A*).—In valuation for the purpose of forced acquisition, or, as it is commonly called, condemnation, the provisions of our National and State Constitutions set forth the principles to apply, and no one questions the privilege of sovereignty to exercise the right of eminent domain when such exercise is found to be for the general welfare. The long-established rule that the owner is entitled to the fair value of his property, unaffected by past or proposed legislation to the benefit of the purchaser, has been interpreted to mean that he is entitled to receive the price on which a willing purchaser and a willing seller would agree, for the property to be conveyed. It may be justly questioned whether this is in all respects a fair treatment of the owner, who may not be a willing seller, preferring to any other the particular industry in question as a matter either of choice or of sentiment. Whoever does not recognize the force of sentiment is unfit for citizenship in either a democracy or a republic. Charles Kingsley, as an Englishman, expressed a doubt of its existence as applied to inanimate concepts, and Edward Everett Hale gave an American's answer in "The Man Without a Country." To him, bereft of sentimental association by the appropriation of the result of the labors of perhaps a lifetime, compensation for sentiment is due, and must justly become an item of value recognized in the exercise of the right of eminent domain.

(*B*).—In valuation for the purpose of sale or purchase there again can be no question of the right of procedure, and, as both parties are interested in obtaining the same information, the rules of accounting are to be followed naturally, and the result arrived at should be the price on which the willing seller and the willing buyer can agree.

(*C*).—In valuation for the purpose of taxation the result is influenced by other conditions than in the two former cases. For many years it has been a principle of taxation in America that taxes should be levied only on physical properties, and it is as yet in only some of the States that a deviation from this principle exists. If it

Mr. Williams. were not for this restriction, the value for the purpose of taxation should be the same as for purchase or sale.

(*D*).—In valuation for the purpose of issuing or limiting the issuance of securities, there is first encountered a question of right to value. So far as the owner's inquiry into the value of his property for his own information, or for that of the proposing purchaser of its securities, is concerned, there can be no question; as to the right of sovereignty to step in and demand or pass on such a valuation, there may be grave question. Waiving for the time the consideration of such right, however, there appears to be no reason for this valuation to be different from that for purchase and sale. Indeed, the two cases are similar in all respects, for the bondholder may be conceived of as buying the plant in whole or in part, and as allowing to the owner for its operation and maintenance so much of its income as exceeds a certain stipulated sum.

(*E*).—Finally, the right to a valuation for rate-making by the State, it must be admitted, is only to be derived by an extension of the powers of Government never contemplated by the founders of our commonwealth, and if by the term "investment" the Committee means financial investments, the speaker most emphatically takes issue with the statement on page 11 of the report that "it would be entirely just and equitable, and it is highly desirable to provide by law that future public service properties should be valued on the basis of their actual reasonable existing investment, and to determine or limit rates upon such a valuation if the service rendered permits," and he further emphatically denies that there is either justice or equity, or even common sense or practicability, in the proposition therein advanced.

As an example of the inadequacy of cost to determine value, consider the case of a horse about to enter a race for which a purse of several thousand dollars is offered, and that his ability to win the race is assured. Would any one say the value of that horse at that time was represented by the cost of siring, foaling, raising, and training him, plus, if you will, an allowance for deterioration of his dam?

Following the suggestion of the Committee to its logical conclusion, it results that the illy devised, illy designed, expensively constructed, and extravagantly operated plant is put on a par in earning capacity with one prudently devised, well designed, carefully constructed, and economically operated. In other words, the Committee omits that greatest element of value, in all business enterprise, that element without which there could be no variation in value in the same or similar situations, that element which lies at the foundation of all progress, brains. Of all persons on earth, the engineer should be the last to overlook brains. Brains, and brains only, differentiate

him from the laborer in the street, and our patent laws recognize the compensation due to brains. Mr. Williams.

Take the first entry of brains into a proposition, its conception; if the idea be novel and the business developed on it be successful, then the idea was valuable and is a part of the investment. Let it be a case of a water power; some one conceives the development, some one works out the general scheme and, for this service, usually receives or retains a portion of the corporate stock for which he makes no other payment. In the search for tangible property, behind the investment, this block of stock will be termed "water", but it is really "brains", and is just as much entitled to recognition as a factor in the development as the money actually paid to labor for the construction of the work; but where, beyond the value of the time spent, is there recompense allowed for this item in the scheme of the report?

Again, a corporation secures the services of an engineer who is an expert, and gives him an interest in the property in addition to the fee for his services. He produces for them a plant capable of converting the power of Nature to the use and convenience of man with a higher degree of efficiency than has been accomplished in any previous plant, and besides doing so, he reduces the construction cost per unit of capacity. Are not the brains of that engineer as put into that plant entitled to be counted as part of the investment in that plant, and is it not right and proper that this plant economically constructed and economically operated should be entitled to earn a higher rate of interest than the plant thrown together haphazard, wasting perhaps 50% of the natural energy it is purposed to convert?

These are some of the questions to which one looks in vain for an answer in the Committee's report, and they are the questions for which a formulation of correct principles of valuation must provide.

The programme outlined in the report, and in every discussion of the subject with which the speaker is familiar, puts a premium on bad judgment, bad design, bad construction, and bad management, and a penalty on their opposites. To such a programme neither the engineer nor the public can afford to subscribe.

If a property be only worth its cost, what is the inducement to produce it? Take, as an example, the land needed for a reservoir site. A part is secured at the price of ordinary farm land and the last few parcels at three or four times that price. When all that land has been brought together into one complete whole, is it worth no more than when scattered in isolated parcels under diverse ownership? Is it worth no more as such a whole than the cost of its acquisition?

Is a watch ready to be wound and to run only worth the cost of its component parts and the time of the mechanician who puts them together?

Mr. Williams. The purpose of the public service corporation is to serve. The value of a service has only this connection with its cost, that it must not be less than the cost or the service cannot be rendered, and the compensation to be demanded must not be greater than the value, else no one will accept the service.

To determine properly the rate for a service, its value to the recipient must be established, just as the value of any other article or concept to be marketed must be established.

This is a broad question, a hard question, and it involves the same problems as those confronting the investor every day, that this business may be highly lucrative, while that may be a losing venture, when, in either case, the worth of the physical property has no determining influence on the value of the project.

There are in a certain locality two water-power plants designed to serve the same community. One cost about $300 per horse-power of machine capacity and the other less than $100, on the same basis. In one of them the advantages of expert training and brains were utilized to the utmost, and the plant has proved a model for numerous later ones scattered throughout the country. In the other the brains that entered were used mainly in copying the mistakes of previous designers and in building up a huge physical investment on which bonds could be sold. The latter concern is bankrupt, the former is paying excellent dividends. If the rules laid down by the Committee be accepted, the bankrupt concern would be earning a mediocre return on a bad investment, and the other would have its earnings reduced one-half, while the customers of the one plant would pay three times as much as those of the other for the same service. In its work the Committee, like many of our public service commissions, seems to have started from the wrong premises. Even in dealing with the value of physical property alone, which, by the way, the Courts have usually held to be only an indication of value, there is apparent a misunderstanding of depreciation. The substance of the definition usually accepted for this term is the value of the service already rendered by an article not new. In a valuation, this has nothing to do with the case. So far as its value is concerned, whether the property be a railroad track or a jack-knife, its past service is of no interest; it is its future possibility of service that is involved. The true definition of depreciation as used in valuations then is as follows:

"Depreciation is the difference between the present worths of future service and scrap value of a new article, and of the article under consideration."

If this definition be gotten clearly in mind, it will help in wiping out some of the difficulties contained in the subject of valuation.

GEORGE F. SWAIN, PAST-PRESIDENT, AM. SOC. C. E.—The speaker would say at the outset that he has read this report with greatest interest, and considers it, in regard to everything except the matter of depreciation, the most sane, sensible, and logical discussion on the subject that he has seen. It is clear, forcible, fair, and accurate. It presents the various points impartially, at the same time giving the conclusions which the Committee deems justifiable, and with which, in most cases, the speaker cordially agrees. It brings together a great number of facts and illustrations drawn from experience which have never been collected before, and which cannot fail to be of great value in the future consideration of the subject. The thanks of the Society are cordially due to the Committee for its work. Mr. Swain.

With reference to the question of depreciation, however, the speaker believes that the views of the Committee are lacking in clearness, that they involve confusion of thought, and are misleading. He, therefore, wishes to give his own views on the subject, though he fully realizes that he is quite as likely to err as the members of the Committee. This subject of valuation is one which is in the making, and it must be expected that opinions, both of individuals and of the Courts, will experience modifications as the subject becomes more definitely understood and the principles definitely established.

The question of depreciation is twofold:

First.—What is the actual depreciation?

Second.—What value of the property shall be taken for the purpose in view?

With reference to the actual depreciation, this, as every one recognizes, is not a matter of computation or theory, so much as of actual inspection. The actual depreciation will depend on the maintenance. If a steel bridge has been kept well painted and in good repair, it may not have depreciated at all; that is to say, it may be exactly as good as new, although, on account of obsolescence, or, in this case, the fact that increasing weight of rolling stock may require a new structure before the old one is worn out, it may be less suitable for present purposes than when built.

Actual depreciation, therefore, depends on maintenance, and is to be determined largely by inspection.

With reference to the value of the property which is to be taken, the speaker is glad to know that the Committee definitely states that this value depends on the purpose in view, and may differ according to that purpose. It is true, of course, that a public service corporation would generally be willing to have the same valuation used for taxation as for rate-making, provided a fair value for rate-making purposes is allowed, because the rates should be adjusted so that a fair return will be allowed to the owners after taxes have been paid. This being

Mr. Swain. the case, it is evident that it can make no difference to the company how high the taxes are, if such fair return on a fair value is allowed. So far as physical property is concerned, however, a valuation for taxation would not necessarily be the same as for rates. One instance alone will illustrate this point. The construction of a railroad station may have required the taking of a piece of city property with the buildings on it, the latter having been afterward destroyed. A fair valuation for rate-making would include, of course, the cost of the buildings, but these having been destroyed, there is no taxable value to represent their cost except an intangible value. If, therefore, the same value is used for rate-making as for taxation, then, so far as taxation is concerned, this value is not that of the physical property alone, but includes an intangible element.

Coming now to the main point which the speaker wishes to discuss, the burning question with regard to the value which is to be taken as the physical value for purposes of rate-making, to which the report of the Committee is limited, is the question whether a fair return should be allowed on the cost-of-reproduction-new, or on the cost-of-reproduction-new-less-depreciation. With regard to this, the report of the Committee states definitely in several places that the cost-of-reproduction-new-less-depreciation should be used. For instance, on page 10, it is stated:

"The result obtained after making deductions for obsolescence would be the value of existing items of property new, which should be further diminished by the net amount of the depreciation of the existing property."

Again, on page 19, the report states:

"In order to complete the valuation by this method [reproduction-new], it is necessary to make a deduction from the values obtained as above stated, for the net accrued depreciation of the physical property due to age and other causes."

Once more, on page 32, it states:

"A depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property."

Again, on page 72, the distinct statement is made: "The depreciation value of property should be basis for rate-making."

However, in the Addendum to the report, page 3, it is stated:

"That there are certain methods of compensating for the loss of value due to the depreciation of perishable property, which require that the property be valued as if new in order that injustice may not be done to the owner of the property, but that there are other methods

of compensating which require that the property be valued at less than its value new, or, in other words, at a depreciated value, in order that injustice may not be done to the public." Mr. Swain.

In other words, the Committee definitely states here that whether the cost-of-reproduction-new or cost-of-reproduction-new-less-depreciation shall be taken depends on the method of depreciation allowed.

In the speaker's opinion, the method of depreciation to be allowed is an accounting problem, the object being simply to set aside out of earnings each year such sum as may enable all renewals in kind to be made without any increase of capital.

The speaker contends that a depreciation allowance may be considered, not, as the Committee states, as "a payment from the rate-payer to the corporation of a part of its investment", but that it is a payment of the same kind as one for maintenance. It is a payment to ensure the permanent maintenance of the plant in kind without increase of capital. Moreover, it is a payment to the corporation and not to the stockholder who has invested the money. It is a payment which should not reach the stockholder, but should be held by the corporation for a specific purpose, *viz.*, the indefinite perpetuation of the property.

In general, the proper method of allowing for depreciation appears to be clearly the sinking-fund method, because that is most favorable to the public and equally favorable to the corporation.

The Committee is perfectly right in stating (Addendum, page 3): "that the owner of a public utility property is entitled to have the rates made high enough, under normal conditions, to pay for the lessening value of the perishable property, so that when such property goes out of use he shall have received, not only a fair return for the use of the capital invested in such property, but shall also have the capital invested in it paid back to him," except that the original capital should not be paid back to the investor, but should be held by the corporation for renewal when the time comes.

In other words, capital honestly and properly invested should not only receive permanently a fair return, but the rate should be high enough to return to the company an additional amount each year, which, if safely invested, will amount, when the property requires renewal, to the cost of renewing it in kind, that is, as it was originally, without any additions or betterments.

The sinking-fund method, as previously stated, provides for this with the least payment from the rate-payers. The latter pay each year, not only enough to give a fair return on the capital, but a sum which, if placed at interest, will amount to the cost of renewal when renewal is necessary. These payments for depreciation by the rate-payer should be allowed to earn interest, and if so allowed, the yearly payments will

Mr. Swain. be smaller than if not allowed; the rate-payers are just as much entitled to claim that interest should be allowed on their payments for depreciation as the owners of the property are entitled to claim that they should be allowed a fair interest on their investment. All that the company is concerned about is that it should have funds in hand, obtained from rates, which will enable it to pay for renewals as they are necessary. All that the rate-payers are concerned with is that they shall have contributed each year the smallest sum necessary to bring this about, and this is the sinking-fund depreciation.

Now, when the rate-payers have paid rates high enough to allow a sinking-fund depreciation, they have no further interest, as rate-payers merely, in the question, nor is it of any consequence to them, as rate-payers, what the company does with the depreciation allowances which they have paid. It makes no difference to them whether the company puts these depreciation allowances in a fund, whether it uses them for extensions to the property, or whether it distributes them as dividends. Having contributed these sums, they are entitled to claim, and should claim, and the Courts will sustain them in claiming, that when the property or item under consideration is worn out, it shall be replaced by the company at its own expense, that is to say, without further contribution from them, as it was before, without any increase of capital on which future earnings are to be made. The rate-payer, of course, is concerned that the depreciation funds should be safely invested, so that they will be available when the time for renewal arrives, and, of course, it is for his good that the rate of interest allowed in figuring the depreciation allowance by the sinking-fund method should be as high as is safely practicable; but as long as the company must replace every item of its property in kind, as it wears out, at its own expense, and is in a position to do so without injury to the service rendered, the rate-payer has no further interest with what is done with the depreciation allowance. As a citizen, or as an investor, he may be concerned in this matter, and would object to having the depreciation allowances paid out in dividends, as this course would enable the stockholders to receive dividends greater than a fair return and then, perhaps, sell the property on the basis of such exaggerated returns to other investors who might not realize that the former owners had made no provision for renewals. Proper accounting and financing, therefore, require that only a fair return shall be paid in dividends, and that the depreciation allowances shall be invested or held in some manner which will insure their being used for the purpose for which they were paid in. This is for the interest of stockholders rather than for the interests of the rate-payers, as such.

It will probably not be questioned, therefore, that the stockholders of a property built to-day are entitled to a fair return, permanently, on the money originally invested, if honestly and properly spent, and that the rates should also provide for the accumulation of yearly depreciation allowances. If this is so, is it not then clear that the cost-of-reproduction-new should, so far as a physical valuation is concerned, be used as the basic value on which earnings should be secured? In order to answer this, it is at this point necessary to see clearly the relation between the reproduction method and the cost-new method. Mr. Swain.

In a concern organized to-day under public supervision, with accounting methods also under supervision, there would never be any necessity for using the reproduction method or for making any valuation. Its stockholders would be allowed to earn a fair return on the cost, plus depreciation allowances; renewals would be made in kind without increase of capital, and all betterments would be charged to capital. If there should be a lack of a fair return in early years, any such deficiency would be included under cost of developing the business, and in subsequent years a return would be allowed on this also. Such a procedure would not prevent an unwise venture from failing, for an increase of rates might diminish net earnings, there might be a continued cessation of dividends, and the concern might eventually collapse; but a concern wisely planned and economically built would prosper, and its stockholders would have no cause to complain. There would be no questions as to the appreciation of land; if, through the growth of the community, the value of its lands should largely increase, the concern could not be bought out by another concern with the expectation of fair return on the increased value, because the original cost would govern.

In the case of most corporations, however, especially railroad corporations, the original-cost method is not possible, because the data are not available. What is sought now in making a valuation, is a basic physical value from which to start new and on which, in future, to base earnings. For this purpose no other method is available than the reproduction method. The cost is estimated of reproducing the property new at the present time. This is taken as the basic physical value of the property, just as the original cost, with subsequent betterments, would be taken as the basic value if that method were available. The speaker does not understand that it is at all contemplated that the reproduction method shall be applied from time to time to the same property and the rates modified according to each valuation; he does not believe in such a procedure. As he conceives it, a valuation is to be made once for all in order to get a basis from which to start new, after which, proper accounting methods will be required, renewals in kind must be made from earnings, better-

Mr. Swain. ments capitalized, and rates can be regulated in the future without another valuation. The reproduction method is used because it is the only possible method of general application.

The kernel of the whole matter, therefore, it seems to the speaker, is the question whether the cost-of-reproduction-new should be taken and used as the equivalent of original cost. Probably few will question the statement that, when available, the original cost, allowing for deferred dividends, is a proper basis of physical valuation on which earnings should be secured. Stockholders who invest their money honestly and intelligently in a public utility concern, are entitled to receive a fair return on that cost permanently, together with operating expenses, taxes, and a depreciation allowance, and any deficiency of dividends is entitled to be capitalized. Depreciation should not be deducted in finding a physical value on which to base proper earnings. The original cost is the proper basis, and is the basic physical value on which stockholders should receive a fair return.

Now, if the cost-of-reproduction-new is also a proper basis, it must give the same result in cases where both are applicable. In the case of a new concern, the original cost would be, of course, the cost-of-reproduction-new. The same would be true in a concern in which the cost-new could be ascertained, and the books had been properly kept, and in which costs had not altered since construction. There are many corporations, however, in which the original cost method is impossible of application by reason of lack of data. In this dilemma, as the speaker understands it, we turn to the cost-of-reproduction as the only other available method. In using this method we are necessarily thrown largely on surmise and judgment, but no other method is possible. The costs have changed since the property was built, and no one can say with certainty that the cost-of-reproduction-new will be greater or less than the original cost. In the case of our railroad companies, lands were granted them by the public, and these, together with lands which they bought, have largely increased in value, mostly by reason of the existence of the railroad itself. Lands owned by private owners have likewise increased in value. It seems to be no injustice to allow the corporation the benefit of an increase in value which has come from its own existence, and which is no greater than the increase in value which its existence has brought to the property of private owners. Moreover, in the case of many of our railroad companies, if not of all of them, dividends were deferred for many years, reorganizations have taken place, and a great deal of money has been spent in betterments and charged to operating expenses; so that, as stated, the cost-of-reproduction-new cannot be said to be greater or less than the original cost, if the latter could be accurately ascertained.

The statement then is this: The reproduction-new method gives the same result as the original cost method where both are applicable; in

this case the original cost is a proper basis and would be used. Must we not, therefore, in cases where the original cost method is not applicable, assume the cost-of-reproduction-new to give the same result and to be used in the same way as the original cost would be if we could find it? The answer to this question appears clearly to be in the affirmative. Mr. Swain.

The Supreme Court, in the Knoxville decision, held that if, in the past, rates had not been high enough to provide a proper return and a proper depreciation allowance, "the true value of the property * * * cannot be enhanced by a consideration of the errors in management which have been committed in the past." This rule should work both ways; and if, in the past, rates have been so high as to have permitted excessive profits, or if the public has given lands for nothing, then the value of the property cannot be reduced by a consideration of the errors committed by the public in "the past", even if we choose to designate as errors a policy which, on the whole, has resulted in a wonderful national development and prosperity. We are starting new with a value of the property, and in the speaker's opinion, there is no method which will be uniform in application, and will reconcile all differences of conditions, except to take the cost-of-reproduction-new, and to consider that this is to be used exactly as if it were the original cost.

The first principle on which the speaker has based his conclusions, therefore, is that the cost-of-reproduction-new is to be taken and used in the same manner as the original cost. If this be true, it is perfectly clear that the undepreciated value should be the value on which a fair return should be based, just as the original cost, when available, should be the fair value on which a reasonable return should be based.

Of course, it must not be forgotten that a physical valuation is only one of the elements to be considered in deciding on rates or any other question. Neither the original-cost alone, nor the cost-of-reproduction-new alone, is necessarily the present value; but we are not discussing this question here. We are simply endeavoring to decide what physical value is the proper one to consider as the basic one on which, so far as physical value is concerned, fair earnings are to be made.

The Committee has apparently been governed in its attitude by the assumption that the Courts had finally and definitely decided that the cost-of-reproduction-less-depreciation must be used, and its endeavor has been directed toward finding a method by which the same total annual return would be received by the corporation as by using the original cost.

This result is accomplished by the Committee's method only in case the rate of return on the investment and the rate of return on the sinking funds are the same. If the sinking funds are invested in extensions of the plant, this may be true, but in many cases the sinking

Mr. Swain. funds cannot be invested in the plant, and the rate of return on the sinking fund would be less than that received from the investment. Indeed, inasmuch as the sinking fund must be assumed to be invested safely above all things, there seems to be no reason for assuming a greater rate of interest on it than the company receives for its cash, or the rate of interest for time loans, which, in general, would be less than a fair return on the investment. In this case, the Committee's method, as shown by Column 7 of the table on page 34 of the report gives a smaller return, except in the first year, than is fair. It results, therefore, in confiscation.

The object of the Committee, however, is perfectly clear and quite praiseworthy. It assumes that two things have been finally and irrevocably settled by the Courts:

First.—That the cost of reproduction-new-less-depreciation must be used as a basis;

Second.—That the mistakes of the company in the past, in not charging rates sufficient to provide a fair return plus a depreciation allowance, cannot now be rectified.

These two findings are justified in the decision of the Supreme Court in the Knoxville case. It must be remembered, however, that the Courts, being human institutions, are subject to error; that every Court has had its decision reversed probably more than once; and that the Supreme Court of the United States has reversed its own position in a number of instances.

Much is said, in discussing this subject, as to respect for the Courts. The speaker believes that there is no one who has greater respect for them than he, but it is very important to understand clearly what respect for the Courts implies. It certainly does not imply agreeing in all cases with their decisions, or even accepting them without argument, if we believe them wrong. Judges, like those who are judged in this world, are human, and it would be less respectful to attribute to them superhuman attributes by assuming them to be always correct and infallible, than to admit their fallibility. Proper respect for the Courts means that we should believe our judges to be honest, faithful, disinterested men, honestly striving to administer justice, and that we should accept their verdicts when rendered, if we have to and are obliged to act on them; but that does not prevent us from fighting with all our might and main to convince them that they are wrong, if we believe such to be the case. The present subject is in the making, and it is quite possible, if not probable, that the Supreme Court may reverse its finding with reference to the facts referred to. Assuming those findings to be correct, however, the Committee pursues a perfectly consistent course. This may be illustrated graphically:

In Fig. 8 let OY be the first cost of a piece of property, and OX its life, say, 10 years. The curve of the sinking-fund depreciation is shown by the curve, YX. ab is the annual payment which in 10 years will amount with interest to the original cost, or the assumed cost of reproduction. At the end of 5 years, five annual payments have been made, which, with interest, amount to $a_5\ b_5$, leaving $b_5\ x_5$ as the assumed depreciated value; $c_6\ b_6$ is equal to ab plus the interest for a year on $a_5\ b_5$; $c_6\ b_6$ is also the annual payment which in 5 years with interest will amount to the depreciated value, $b_5\ x_5$. Now, the Committee argues, assuming that the depreciated value must be taken, that the company should be allowed to earn a fair return on $b_5\ x_5$ plus an annual depreciation charge of $c_6\ b_6$, which is the same thing as a fair return on $b_5\ x_5$ plus sinking-fund interest on $a_5\ b_5$ plus the depreciation charge, ab. If, therefore, the rate of return on the property is Mr. Swain.

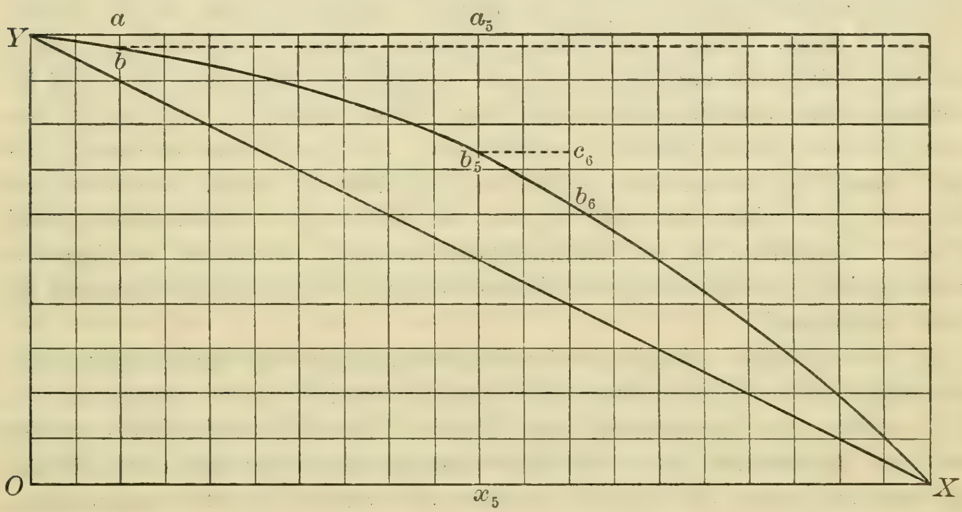

FIG. 8.

equal to the sinking-fund interest, this is the same thing as a fair return on OY plus a depreciation charge, ab. If, however, as must be assumed the fact, the sinking-fund interest is less than a fair return on the investment, then the Committee's method does not allow a fair return on the original property plus a proper depreciation charge.

If, therefore, a concern built to-day, and costing OY, is appraised 5 years from now, and if, on account of the original records being lost, or for any other reason, the cost-of-reproduction method is used, it will result in not allowing a fair return to the investor on his investment. It must be reiterated that though the rate-payers have contributed the sums which go into the depreciation fund, the stockholder has not received them; the company has received them, but it is obliged to keep them available for a specified purpose, namely, for reproducing the property when it is worn out. The statement of the

Mr. Swain. Committee that inasmuch as the sum, $a_5\ b_5$, has been returned by the rate-payers, the company is not entitled to receive a further return on this property, is an evident fallacy, for though $a_5\ b_5$ may represent what has been paid by the rate-payers, with interest, it has not gone to the stockholder, and interest on this sum is not entered in the capital account, but in the reserve account.

Moreover, even supposing the Committee is justified in believing, or hoping, that the Courts would allow a fair return on $b_5\ x_5$ plus a depreciation allowance, $c_6\ b_6$, is it not equally probable that the Courts will require the latter to be held in reserve and not distributed as dividends? In this case the property of the stockholder is confiscated.

Moreover, if a depreciation allowance of $c_6\ b_6$ is permitted, this will allow the replacement of the property when worn out, at the point, X, only its depreciated condition, represented by $b_5\ x_5$ and new capital will be required equal to $a_5\ b_5$, in order to replace it new, as, of course, would be done. The stockholder, of course, must be allowed to earn a return on this new capital, $a_5\ b_5$, and thereafter; therefore, it would be earning a fair return on the total cost of the item, OY. What, in this case, is to be the depreciation allowance? If it is allowed to remain, as the Committee recommends, equal to $c_6\ b_6$, then the total return will be too great, for when the item is new the depreciation allowance should be ab. According to the Committee's method, therefore, in fairness to the public, the depreciation allowances must be changed whenever an item is renewed.

If, on the other hand, the Courts should allow the total return recommended by the Committee, and should permit it to be distributed in dividends as the corporation may decide, then the stockholders would receive a fair return and a fair depreciation allowance, but only in case the rate of return on capital, and the rate earned by the depreciation fund, were identical; if the latter were less than the former, as must be assumed, the property of the stockholder would be confiscated, for it would not receive a fair return.

All this uncertainty and complication will be avoided by allowing a fair return on the cost-of-reproduction-new, and a fair depreciation allowance on this same sum (namely, ab), which will renew the item, if new, in the term of its life, independent of its age when the valuation is made.

It seems reasonable to assume that the Courts will finally see the truth of this, if we insist upon it, and make it as clear as we can.

It is true that the method recommended by the Committee is fairer to the corporation than that which has often been followed. It is much fairer for the corporation to be allowed (referring to Fig. 8) a fair return on $b_5\ x_5$ plus a depreciation allowance of $c_6\ b_6$ than to be allowed only a fair return on $b_5\ x_5$ plus a depreciation allowance of ab. But the speaker maintains that the investor is entitled to

a fair return on OY plus a depreciation allowance, ab. He believes that the Committee loses sight of the fact that the question is not that of following a piece of property throughout its life, but of jumping in at some intermediate period, as at x_5, with no knowledge of original cost, and determining at that time the value on which fair returns should be allowed. Mr. Swain.

The Committee, in its illustrations in the Addendum, assumes that in each year precisely the depreciation allowances are invested—no more and no less—in extension of the plant. Refer, for instance, to Tables 5 and 6. Table 5 represents the sinking-fund depreciation invested in the plant, and the value-new is given in the last column. The Committee says:

"It would be grossly unjust to the rate-payers to permit the owner of the property, under the conditions presented, to earn dividends, not only on the \$100 000 which he has invested, but also on the additional \$177 609 [at the end of the ninth year] contributed by the rate-payers, and yet this would be the result of using value new in connection with the Equal-Annual-Payment Method of providing for depreciation."

This statement the speaker considers entirely incorrect, with the understanding that the company must renew an item, when worn out, at its own expense without increase of capital. It is entitled to earn a fair return on the items in the last column plus an original depreciation allowance, and at the end of the tenth year, under these circumstances, it will have earned enough to enable it to replace the original plant at its own expense, and will have accumulating funds which will replace the later investments when they require replacement.

By the Committee's method, in Table 6, at the end of the tenth year, the company replaces the original plant with new capital, which is contrary to the fundamental principle of accounting and contrary to the rules laid down by the Interstate Commerce Commission, and indeed contrary to all sound principles of financial management. Of course, it is easy to see what the Committee has done. It has assumed that the original plant has been exchanged for the new plant which has been built from the depreciation allowances, and that instead of selling these additions and putting the money into replacement of the original plant, it simply invests its new capital in renewal. This illustrates, however, in the speaker's opinion, the confusion which exists in the report; the illustrations are ideal, not real; theoretical, not practical. No concern would be able to invest each year, beginning with the second year, precisely the depreciation earnings in extensions. Indeed, it is illogical and misleading to confuse the question by assuming that depreciation earnings are invested in the plant. So far as the rate-payer is concerned, it makes no difference what is done with the depreciation earnings. If we can decide correctly how to treat the

Mr. Swain. original plant, we shall then know how to treat any additions to that plant. The original plant should be kept separate in our consideration, and the fundamental principle must be maintained that it must be renewed by the company, when worn out, at its own expense. The Committee's method of treating the subject is misleading and confusing in this respect.

The speaker has taken Table 5 and extended it, showing what the result would be, if, as he maintains, the company should be allowed to earn a fair return on the value new plus a depreciation allowance. The result shows that at the end of 10 years there is an amount available which will be sufficient to replace the original plant without new capital; it also shows additional sums equal to the proper depreciation allowances necessary to replace the extensions which have been paid for from depreciation earnings.

Much has been said about the depreciation fund, and the question now arises, how large a fund is necessary in the case of an ordinary public service corporation. Such a corporation, after a few years of operation, settles into what may be called a state of financial stable equilibrium. Just about the same sum will be needed each year for renewals, and the only depreciation fund, or reserve, that is necessary is one which is large enough to take care of the yearly inequalities in these renewals. A railroad company, for instance, with hundreds of thousands of ties, thousands of tons of rails, thousands of bridges, stations, cars, locomotives, and other items of property, requires practically a nearly uniform sum each year for maintenance and renewals. In one year it may have to renew more bridges than usual; the next year it may have to renew less, and so with the other items. A comparatively small sum is all that is needed as a depreciation fund or reserve, and the larger the property—that is to say, the more numerous the individual items—the less the fund will have to be. This fund is simply in the nature of a reservoir to balance annual inequalities, exactly as the reservoir for the water supply of a city is required to balance the unequal draft at different hours of the day or in different months of the year. The more uniform the draft and supply, the smaller the reservoir may be. In other words, the depreciation allowances set aside for any given item of property in any given year are at once paid out to renew other items which require renewal in that year.

The speaker wishes also to call attention to another source of confusion in the report of the Committee, and one which is also to be found in many reports and decisions. It arises from confusing two meanings of the word "depreciation". Properly understood, depreciation means a loss of value due to wear and use, or to approaching obsolescence. This is the kind of depreciation that we have

been considering. The word is often used, however, to mean also the lessening of value due to the fact that a given piece of property might be produced to-day at a less cost than it could be produced for when new. This is an entirely different thing, and the same term should not be used for it. Mr. Swain.

When it is claimed that the cost of reproduction new should be used as a basis for rates, the reply is often made by those who have not considered this distinction: "That cannot be correct; you allow the appreciation of land because it has increased in value; why should you not allow the depreciation of those elements which have decreased in value; one is the opposite of the other; you should allow both or neither."

This argument is fallacious, as will clearly be seen by considering the foregoing distinction. The reason we allow the appreciation of land is because we are using the cost-of-reproduction method, and, in using that method, we do allow for "depreciation of things which have depreciated". That is to say, every element of property which costs less to produce to-day than it did originally we put in the inventory at its cost to reproduce to-day, just as we put in the land at its cost to reproduce to-day. Items of property which cost more to reproduce to-day than they did originally we put in at their present cost; items which cost less to reproduce to-day than they did originally we put in also at their present cost; we make no difference. The question of depreciation due to wear has nothing to do with this; it is an entirely different one. Instances of this fallacy will be found in the Committee's report, and also, for instance, in the decision quoted by the Committee on page 51, which, instead of being clear, as the Committee states, the speaker considers to be quite confused in its reasoning. For instance, on page 50, the Committee says:

"The Courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value. The principle involved in the two cases is the same, and there is no reason to think that its application to both would not be supported by the higher Courts."

The speaker respectfully contends that the principle involved in these two cases is not the same, but is entirely different.

The question of depreciation due to wear, then, is not to be set in contrast with the appreciation of land, and it is this depreciation which we are considering. If we consider the cost-of-reproduction method, we must put in every element, land included, at the fair cost of reproducing it to-day, whether it is greater or less than its original cost.

Mr. Swain. In view, therefore, of the arguments adduced, there is no question in the speaker's mind that in valuation for rate purposes the undepreciated value should be taken, unless we are ready to admit that the cost-of-reproduction-new is not to be taken as a figure which replaces the original cost, which cannot be ascertained. If, however, we assume that the reproduction value is not to be taken in place of the original cost, then we find ourselves in the logical inconsistency, when we use the reproduction method, of using a method which does not give the same results as an admittedly correct one (namely, the original cost) in cases where both are applicable.

TABLE 24.

(1) Year of operation.	(2) Percentage on investment earned for distribution to investors and for replacements and renewals.	(3) Percentage paid to investors.	(4) Percentage remaining for replacements and renewals.	(5) Depreciation in percentage.
1st	5.00	5.00	0.00	4
2d	5.75	5.75	0.00	4
3d	6.50	6.50	0.00	4
4th	7.25	7.25	0.00	4
5th	8.00	8.00	0.00	4
6th	8.80	8.00	0.80	4
7th	9.60	8.00	1.60	4
8th	10.40	8.00	2.40	4
9th	11.20	8.00	3.20	4
10th	12.00	8.00	4.00	4
Total for 10-year period	84.50	72.50	12.00	40
Average for 10-year period	8.45	7.25	1.20	4

Table 24 will make clear the fact that to take the cost-of-reproduction method, using the value-new-less-depreciation, may, in some cases, mean confiscation. Suppose, for instance, that a public utility property is constructed and put into operation—let us say, an interurban street railway. Suppose that all the money is expended properly and that the road competes with a steam road and charges rates, say, 10% less than the latter, in order to attract business. Suppose there is no need of expenditure for renewals for the first 10 years, after which renewals begin, and soon attain a state of stable equilibrium. Suppose that the percentage earned on the investment which is available for distribution to investors and for replacements and renewals—that is to say, the amount earned above operating expenses and taxes—is as shown in Column 2, varying from 5% in the first year to 12% in the tenth and subsequent years. Suppose that investors receive the percentage in Column 3, varying from 5% in the first year

up to 8% in the fifth year and in each year thereafter. Then the percentage remaining for replacements and renewals will be as shown in Column 4, being zero for the first 5 years, and increasing to 4% in the tenth year and thereafter, the total accumulation in 10 years being 12% on the capital. Suppose that the depreciation is calculated by the straight-line method, taking a life of 25 years, or 4% per annum, making 40% in the 10 years. Mr. Swain.

This property, then, at the end of the 10 years, is earning 12% and gives every indication of continuing to earn that amount, of which 8% is paid to the investors and 4% remains for replacements and renewals, in addition to the fund of 12% which can be drawn on for the same purpose. The property is successful, and although the original cost was $1 000 000, its stock sells for about 120; that is to say, is bought and sold between investors on the basis of $1 200 000.

Now, suppose that, for some reason or other, the original records have been destroyed, perhaps by fire, and that an appraisal is to be made at the end of the tenth year. The cost-of-reproduction method is used and shows a cost-of-reproduction-new equal to $1 000 000; but the commission says we must take the depreciated value, and the depreciation is about 40%, so that as a basis for earnings, the value of the property is only $600 000. To this we may add the 12% in the renewal fund, or $120 000, making a total of $720 000. A similar result is reached if we use sinking-fund depreciation. The original stock paid 8% and called for a distribution of $80 000 annually. If 8% is now only to be allowed on $720 000, it will call for $57 600 annually, or 5.76% on the par value.

Under these circumstances, the value of the stock in the market will drop from 120 to perhaps 80 or 90, and if there is danger that some public authority will take the property on the basis of a value of $720 000, the stock may drop to 72, or even lower. In other words, the value made on the basis of cost-of-reproduction-new-less-depreciation, would confiscate the property of the stockholders, which was selling in the markets at $1 200 000 and justified that value, and now becomes worth only about $800 000, or perhaps less. If this is not confiscation, then the speaker does not know what confiscation is.

This issue must be faced. The Committee, in the table on page 34 of the report, shows a progressively decreasing return on the value of the property, and, on page 49, it says:

"It is true that of the return on a given item of property, the part applicable to dividends becomes less as the property grows older, but this is as it should be; a part of the capital invested in that item has been returned and has either been invested in additions or replacements, or temporarily in a depreciation fund, so that the whole of the original investment is at all times earning returns applicable to dividends".

Mr. Swain.

As has been already demonstrated, it is only when the rate of return on the property, and the rate of return on the depreciation fund, however used, are the same, and if the depreciation allowances may be divided, in part, among the stockholders as dividends, that justice will be done; otherwise, there will be confiscation.

This report will be understood to mean, the speaker believes, that dividends should decrease progressively, or that, if a valuation is made of an admittedly successful and honestly financed concern, like the one used in the previous illustration, then dividends must be reduced on the basis of that valuation. Does the American Society of Civil Engineers, or a Special Committee of the American Society of Civil Engineers, wish to advocate such a procedure? And if so, for what reason? Merely because the Courts may have made mistakes in the past; and to conform to an ideal and unreal point of view taken by the Committee. This report will postpone the day when public service commissions, Courts, and the public will perceive that a continuing concern, honestly serving the public at fair rates, is entitled to dividends equal to a fair return on the full amount of the investment; and by assuming, as a part of the argument, that all depreciation reserves are invested in extensions, it will obscure the fundamental principle that property which wears out should be replaced by the company in kind at its own expense.

If thus interpreted, it may do great injury to public utility companies, and so to the public at large.

There is one quotation which the speaker wishes to get into this discussion somehow, if Mr. Humphreys does not include it in his remarks. It is from a brief by Charles F. Mathewson, of the New York Bar, in the case of the Kings County Lighting Company *vs.* The Public Service Commission, and Mr. Humphreys says it is the most logical statement he has read. Mr. Mathewson says:

"The proposition [to deduct 'accrued depreciation' in valuing plants in rate-making] is so absurd on its face that it hardly needs discussion to show its fallacy. Why, aside from the question of 'confiscation', should consumers, for exactly the same service, equally efficiently rendered, expect to pay less in the sixth year than in the first year, merely because some items of plant will (viewed at the sixth year) require replacement at a date in the future then nearer than such date was at the beginning of operation? As well might it be claimed, to repeat a homely illustration, that a farmer should regulate the price of the eggs which he sells, by the age of the hen which lays them—reducing the price of the product as the hen gets on in years. The reason he does not is that the service efficiency and operating value of the hen, as evidenced by the quality of the eggs which she lays, are not impaired by the fact that her life is advancing. That advancement may concern the farmer and possibly concerns the hen; but it in no manner affects the value of the eggs to the consumer, or justifies

him in demanding them at a lower price than he paid at an earlier period of her life. The consumer of the eggs must expect to pay a sufficient price to afford a return to the farmer on his total investment in the hen during her life, *plus* enough more to enable the farmer on her death to replace her, and thus keep his investment unimpaired. A farmer could hardly be expected to invest in hens for the purpose of supplying the public with eggs, if for a portion of their life he was to receive a return on only a third or a half of his investment; and any such rule would simply compel the public to go without eggs until the regulating power (if such there were) saw fit to revise its reasoning. There is absolutely no difference in the economic principles applicable to the operation of a gas plant and the operation of a hennery, so far as concerns right to return on capital; and what is absurd in one case is equally absurd in the other. The fact that the rate of return in the one case is subject to reasonable regulation, and not in the other case, has no bearing on the main proposition." Mr. Swain.

Mr. Mathewson is here arguing against the proposal to allow the stockholder only a fair return on cost of reproduction-new-less-depreciation.

The speaker wishes to say one word in regard to Mr. Snow's statement, in order not to be misunderstood. The speaker did not say that the reserve funds should not be invested in the property, but did maintain, and still maintains, that it confuses the main question to assume that they are necessarily going to be invested in the property. They need not be, and in some properties there is no expansion for many years. It confuses the question to assume that they are going to be thus invested. They must be invested so as to bring a return, and the only obligation the company is under is to replace every element, as it wears out, in kind, at its own expense. The speaker wishes to be distinctly understood that he did not say that it was an improper use to invest the reserve funds in the property; and he also called attention to the fact that, in most public utility corporations, such as railroads and street surface railways, a small fund will provide for all necessary contingencies; that is, for the variation in the renewals from year to year, only a small fund is required, and that is all that need be held.

J. W. LEDOUX, M. AM. SOC. C. E. (by letter).—In regard to the valuation of utilities for the purpose of rate-making, it would be desirable if engineers could agree on general rules which could be followed in all cases with the least injustice both to the owners of these utilities and the public. Mr. Ledoux.

In these questions most people look to Court decisions for their authority, but the Courts have not settled the matter definitely, and they, as well as those versed in the law, cannot be expected to have

Mr. Ledoux. the special training to handle this particular problem as well as technically trained and experienced engineers free from bias.

The writer is of the opinion that the members of the Engineering Profession are freer from injustice than those of most other professions. Unfortunately, however, partisanship does exist, and it is common to find municipalities or private corporations looking for engineers who know how to, and will, make a strong plea for their side of the case. This phase, in an exaggerated form, was illustrated by a prominent engineer who was heard to say recently that "If a consulting engineer, in his public testimony, is known to have given a corporation fair treatment, his 'goose is cooked' forever afterward as far as municipal employment is concerned". This condition tends toward shiftiness and ambiguity in the discussions of some of those whose standard of justice is subservient to their own pecuniary interest.

There are comparatively few points on which engineers should differ, and it is believed that they would be in perfect accord in all the principles if they were agreed as to the premises.

This discussion is written from the standpoint of a water-works plant, and is based on what the writer considers, at the present moment and after careful reflection, to be abstract principles of justice. These conclusions were reached after the successive abandonment of several materially different opinions which, at the time, were held with more or less assurance. He realizes that as now drawn they are not strictly in accord with some Court decisions which necessarily must be the gospel, but, in their final analysis, they would be based on the concensus of opinion as to what constitutes fair dealing.

If a public utility were being taken over by a municipality by the process of condemnation, the rates and revenue being settled, it is evident that the value of the utility should be paid. If, however, a public commission has in hand the establishment of rates for a utility, the question of condemnation being non-existent, then the cost of the utility should be determined, in order to find the proper and just fixed charges. In the case of condemnation, if the municipality has the right to establish its own works, and the present plant is adequate, the cost of reproduction less depreciation plus the value of the established revenue should be the criterion. In the case of establishing rates, the cost of the existing plant should be the criterion—not necessarily what it cost the owners, but what is reasonable and proper under the circumstances in which the existing plant was built and developed. Where there is doubt, the recorded book cost should be the guide.

As to the items making up this cost, there should not be material difference between experienced and intelligent opinion. Those things which cannot generally be avoided in average or ordinary practice should be allowed—the cost of preliminary engineering; legal, finan-

cial, and other expert reports; cost of promotion, organization, surveys, designs, land, rights of way, water rights, title examinations, and management, are some of the items which come in before any work is done. Then there is the cost of construction; liability and other insurance; interest during construction; architectural, chemical, legal, engineering, and other expert fees and commissions; cost of lawsuits; most of which items and more are practically certain to occur. There should be added, also, an item for contingencies to cover things which cannot be foreseen, but which, in some form or other, are sure to occur, such, for instance, as weather difficulties, errors, accidents to plant or materials of construction, delays and extra expense due to various causes, such as non-arrival of material, injunctions, obtaining of permits, changes of grade, caving, settling of embankments and foundations, defective dams or reservoirs, failure of sub-contractor, strikes, and many other things which cannot be specifically foreseen, but which should be allowed for in a general percentage based on experience. There are also the items of financing, underwriting, and general contractor's profit. Mr. Ledoux.

Then when the works are finished and capable of supplying service to the community, it generally takes 10 years or more before enough contracts can be obtained to pay current expenses. During this period of waiting, which is common to the majority of plants, when, also, there may be further improvements, construction, and extensions, which, of course, must be allowed for, the owners would have to pay out much more than they receive, and this deficiency of receipts is a necessary item to figure in the development cost of the plant.

It is often held that the valuation of a plant should be the same whether the question of rates or purchase be considered. Theoretically, the position is unsound, for in the case of rates we have a property which properly represents a given investment, on the basis of which rates should be established to yield a revenue profitable to the owners. In the case of purchase there is a given revenue with a given property to determine its value. This revenue may make the property a losing or a profitable venture, and, manifestly, the value of the property for purchase is directly influenced by this fact.

The proposition that the cost instead of the value should be taken in establishing rates may not be accepted immediately, on the ground that, value being established regardless of cost, if any other basis be taken, the property is subjected either to inflation or confiscation. However, from the moment it is decreed that the State has the right to fix the rates of a utility, its power to fix value follows as an inevitable consequence.

It will be generally conceded that the State should recognize as just such revenues as would pay the owners of a utility a reasonable

Mr. Ledoux. return on their investment, but here we are confronted with another difficulty. The investment of the present owners of the utility may have been based on value instead of cost. They may have paid for the utility, by reason of its existing or prospective net revenue, more or less than the cost of the actual items entering into the creation of the property. If, however, these existing net revenues were in all cases just to the utility and the public, there would be no reason for State supervision of rates.

It may be assumed, therefore, that the State would look on such rates as unjust, that would yield an undue return on the reasonable cost of the existing utility. As illustrative, a municipality invites a concern to establish a water-works, the character of which and the water supply being definitely specified. The necessary investment and the rates are agreed on. After some years, the municipality decides to build its own works, possibly at less cost, and, being freed from such expenses as taxes, licenses, permits, etc., the cost of operation will thereby be reduced. Prior to this decision, the net revenues paid a reasonable return on the investment, but, fearing competition and a deficiency of revenue, the original company sold its property to a new company for half the cost. Then a rate-making commission is established, and is called on to determine rates. As an opposite case, a water-works is established in the same manner. The growth and density of population and the net revenues became so great, coupled with the fact that they own the only available water supply, that the original owners are enabled to sell to a new company for double the amount of their investment; after which the rate-making commission is called in to establish new rates. In both these and all such cases it would seem that the criterion should be the proper and necessary investment to produce the actual property, regardless of past trading and stock jobbery.

As a general proposition, where there is no contract, it would be desirable and equitable if the value of all public utilities were identical with the proper and necessary investment, so, if the rate-making commission is presented with a case in which the rates are so high or so low as to produce revenues which make the value different from the cost, should it not readjust these rates to the proper basis? The existence of iniquity and the investment therein does not establish the right to its continuance.

In the abstract, value varies as revenue, which varies as rates, which should vary as cost, and, consequently, value should vary as cost; but if the commission does not have the right to establish rates, then value varies as revenue, revenue varies as rates; but rates depend on "what the traffic will bear", and not on cost; therefore, in this case, the cost and value have no direct relation.

Heretofore, in cases of condemnation by the municipality, or arbitration of the value of the utility, it has been held with justification that an important element of valuation was "going value", which represented what a purchaser could afford to pay for a given plant with revenue, above what the same plant was worth where the purchaser was obliged to wait for the acquisition of that revenue. In such cases this item is truly an element of value, but, in rate cases, it cannot be considered in the same way, because the rates have not yet been established on which to figure the present worth of the deficiencies of revenue according to an assumed rate of revenue increase. For rate purposes, it would be more proper to call this item "development cost", which may be defined as the necessary net outlay over and above the cost of the physical plant up to the time it is earning a reasonable return on the total investment. The practical outcome of this, however, is the same as that of "going value". Mr. Ledoux.

In rate cases, if one must determine this item by calculation, it will be necessary to cut and try. First, assume this item, a revenue, and a rate of increase. From the fixed charges, operating expenses, and assumed revenues, the present worth of the deficiencies of revenue can be calculated. If that does not agree with the assumed development cost, try another value, until finally there is an agreement. If the assumed revenue is more or less than enough to pay a reasonable return on the calculated total cost, a new value for revenue must be assumed, using the same rate of increase as before. Proceed in this manner until the calculated results agree with the assumption; after which, one may be as close to what it ought to be as though he had guessed at it in the beginning.

In many lines of investigation, the snap judgment of a person having broad experience is more reliable than the most abstruse calculation.

Some argue (and generally with justification) that this element of cost should be based on the actual experience of the plant under consideration, but this plant might be grossly mismanaged, more than ordinarily well managed, or the personnel of the operating force may antagonize the patrons; at any rate, it does not seem wholly proper to base cost on the reluctance or alacrity of the consumers to patronize the utility.

If the foregoing reasoning is sound, then the cost to reproduce the given plant will be taken as the basis on which to determine rates, and not the present rates as the basis of determining value, and, as a consequence, new rates.

Now, if the cost, or what is the same thing, the necessary investment, is the proper basis for the commission to adopt in establishing rates, how is this to be determined?

Mr. Ledoux. There is justification for a method which will wipe out past history entirely, take the plant just as it exists, and calculate what such a plant would reasonably cost to construct and develop up to the present. This method would depend entirely on the appraiser's judgment, and this is where such a committee as that of the American Society of Civil Engineers could formulate certain rules of practice which would be of great aid to the appraiser, and which might become a recognized standard by the commissions and the Courts. These rules of practice should be determined, it appears to the writer, by studying carefully the history of twenty or more representative plants in towns having populations of from 2 000 to 1 000 000, and using the data thus obtained for calculating the uncertain and difficult elements of the plant under consideration. The cost of reproduction of the physical plant is neither difficult nor uncertain, but the estimate of the cost of development is the element that should be derived by the method suggested, and, in fact, the Committee might go farther and formulate some scheme such as the following:

First, estimate the cost of the physical plant and property as it exists, taking average prices which have been current for the past 10 years. To the sum thus obtained should be added certain percentages for each of the following items: Promotion and organization expenses, engineering, financing, legal expense and lawsuits, contingencies, and general contractor's profit, and from this total sum should be deducted the cost of constructions which are obsolete or of no value in the plant. The remainder should be the cost to reproduce the physical plant as it stands; there should also be added a general percentage for errors and constructions which are useless, but which are practically sure to occur and impossible to avoid in the majority of plants. Then there should be added a general uniform percentage of from 15 to 40% for development, and this percentage should be determined from the analyses of the twenty or more water-works referred to.

When these percentages are once determined and recognized as reasonable, this method would seem to be the most satisfactory that can be devised for arriving at the proper cost, and in a majority of cases it will be fair both to the owner of the utility and the people. After the cost is once derived in this way, it only remains for the commission to establish such rates as will produce a revenue sufficient to pay all operating expenses, fixed charges, depreciation, and a reasonable return to the owners of the utility.

Many engineers would object to this method on account of the arbitrary fixing of certain percentages to allow for uncertain elements of cost, but, no matter what allowances are made, they must necessarily be based largely on speculation, and it is an advantage, where speculation cannot be avoided, for all engineers to speculate uniformly, for then we will not be confronted with cases like one which came

to the writer's knowledge a few weeks ago, where, in the valuation of a water plant for establishing rates, three experienced engineers estimated the "going value"; two made it some $30 000 odd, and the third more than $130 000. Mr. Ledoux.

At first sight this method would appear to give the recently established water company an undue advantage by reason of its not having incurred the expenses of development belonging to one established for more than 10 years, but these expenses are sure to occur later, and the rates should be such as would yield the desired revenue when the consumers are connected up at the end of the development period.

The subject is one of great difficulty, and no method can be devised which will not in some exceptional cases be apparently unsatisfactory, but, until some recognized method is adopted fair to the utility, investors will not be willing to embark in propositions of this kind, and the people will be the ultimate losers.

It will be observed that in this discussion no deduction has been made for accrued depreciation. From the moment a plant is built, depreciation, an insidious enemy, begins to rob it of its substance and carries away every year a definite portion of its value. If the owners of the plant were responsible for the robbery, they should not receive credit in valuation for the property thus abstracted, as, for instance, if they sold every year a definite portion of the plant and kept the proceeds for their personal use. On the other hand, if the public, or people, living along the lines, for whose benefit or use the utility was constructed, were responsible for this annual abstraction, then the owners of the plant should be reimbursed by the people. When the plant was built, however, both parties recognized the existence of this enemy and the extent to which his maraudings would inevitably reduce the value of the property each year, and the necessity of maintaining by replacement as required the property thus lost. The money to pay for the cost of these replacements can come from only two sources: the owners of the utility or the people. If it comes from the owners, unless they be the people, there is no return for the money thus expended. Unless the utility be a philanthropy, the owners must be reimbursed at least for all legitimate and unavoidable expenditures, for even then the undertaking is profitless. This reimbursement can only come from the revenues produced by the rates charged the people for the service rendered. As this abstraction of value by depreciation neither reduces the amount of the investment, nor inures to the benefit of the owners, therefore, in valuation for rates, no deduction should be made from the proper cost of the property for accrued depreciation.

It will be asked, how is one to insure that the owners of the plant will provide out of the revenues thus produced an inviolable fund

Mr. Ledoux. for depreciation, instead of pocketing this allowance or paying it out in dividends, and letting the plant gradually go to destruction?

If the revenues produced from the rates established by the commission are large enough to pay operating expenses, taxes, interest, depreciation, and profit on the legitimate original cost of the works, and the owners of the plant do not invest this depreciation fund in the plant, it has lost just so much value or investment cost, which has gone to make the profits unduly large. Consequently, an equivalent deduction in cost should be made when the commission again establishes rates.

The same principle should apply the first time if it is found that the previous revenues have been high enough to pay all expenses, profit, and depreciation, while the plant has been allowed to depreciate with nothing to show for a restoration fund.

For purchase, the case is entirely different, depending on the conditions outlined in the four following cases:

Case 1.—The city condemns, the rates being fixed;

Case 2.—The city and owners of the utility mutually agree to an arbitration, the rates being fixed;

Case 3.—The commission establishes the value of the plant at which it will be taken over by the city, the rates being under the control of the commission;

Case 4.—The city purchases by mutual barter, the rates being fixed.

In each of the first three cases, according to the writer's judgment, depreciation should not be deducted unless the company has been earning sufficient to pay all expenses, including depreciation and a reasonable profit, and no restoration fund exists.

In Case 4 depreciation would naturally be deducted, at least by the representatives of the municipality.

The question of street pavements also leads to confusion of thought: If the works were built when no pavements existed, and these were afterward constructed by the city, should the rates be increased thereby, and, in case of purchase or condemnation, should these pavements be an element to increase the value of the utility? In the case of rates, the municipality has added a feature which increases the cost of maintenance and the final duplication of the piping system and, to that extent, the revenues and rates should be increased.

In case of purchase by barter, where the municipality has no right to condemn, pavements certainly add to the value of the utility on account of the greater cost to the municipality to duplicate the piping system. In case of condemnation by the municipality, or the establishment by the commission of a sale valuation, it is a serious question whether the commission should recognize as just an element of value that was created by and at the expense of the people.

ALBIN G. NICOLAYSEN, ASSOC. M. AM. SOC. C. E.—The proper treatment of depreciation is one of the most important questions in the valuation of public utilities. It is also one that has a tendency to become too involved to be followed easily if presented in general terms, and the speaker will not attempt to do this, although his conclusions were originally reached in that way. It is his intention to show how depreciation affects the value, for rate-making purposes, of the physical property owned by two imaginary companies operating under assumed simple conditions. In spite of the limitations of such a method, it will be found that the conclusions reached will apply quite generally, and, in addition, will be of material assistance when the examination is extended to companies operating under more general and more complicated conditions. Mr. Nicolaysen.

The first imaginary company which will be investigated, is assumed to operate a ferry-boat acquired at a cost of $100, and having a useful life of 10 years. If the company has no other investments, and if it is further assumed that it is entitled to earn 6% on its capital stock, and that the amount in the depreciation fund can be invested in such a way as to earn also 6% interest, then the rates should be high enough to return annually the sum of $13.587 over and above all operating and maintenance expenses, including insurance, taxes, etc., and are to be used only for depreciation and dividend payments.

If this is the case, the company is able to pay annually $6.00 to the stockholders and $7.587 to the depreciation fund. What the stockholders received was 6% on their investment, and what was paid to the depreciation fund, at the end of 10 years, under the assumed conditions, would amount to $100, enabling the stockholders to buy a new boat like the first one and continue operations under the original conditions, or to withdraw from the business with their capital unimpaired.

This shows that the combined dividend and depreciation payment in the assumed case, under fair management, should be $13.587 per year, and it is obvious that this amount is independent of the disposition made thereof. The company, if it so decided, could neglect to establish any depreciation fund, and pay out all the net earnings to the stockholders. This would not mean any increase in the return on the capital. The stockholders would receive 6% on their investment as before, and the amount paid in excess thereof, would be a repayment of capital, which in this way would be entirely returned at the end of the tenth year, when the boat would have no value.

Thus it appears that where replacements are paid for by the stockholders, the annual return on the physical property should be $(A + B)\%$ of its "value new", where A is the rate of interest on capital invested, and B is the rate of depreciation figured on the

Mr. Nicolaysen. sinking-fund basis. This return is the same whether or not a depreciation fund is actually established, and should run undiminished through the assumed life of the property.

The correctness of this statement is confirmed by an examination of Tables 25 and 26. The value of the depreciation fund at the end of each year is shown in Table 25; where no depreciation fund is established, the investment of the stockholders is gradually reduced, as shown in Table 26, which gives the amount of the investment for each year.

TABLE 25.—Sinking-Fund Depreciation. Depreciation Fund Created.

Physical property costing $100 and having 10 years life. Annual net earnings $13.587: Stockholders receive 6%; remainder placed in depreciation fund and earning 6% per annum.

Year.	Capital invested.	Return to stockholders.	Depreciation Fund.		
			Payment at end of year.	Interest during year.	Value at end of year.
1	$100.00	$6.00	$7.587	$0.00	$7.59
2	100.00	6.00	7.587	0.45	15.63
3	100.00	6.00	7.587	0.93	24.15
4	100.00	6.00	7.587	1.45	33.19
5	100.00	6.00	7.587	1.99	42.77
6	100.00	6.00	7.587	2.56	52.92
7	100.00	6.00	7.587	3.17	63.68
8	100.00	6.00	7.587	3.82	75.09
9	100.00	6.00	7.587	4.50	87.18
10	100.00	6.00	7.587	5.23	100.00

TABLE 26.—Sinking-Fund Depreciation. No Depreciation Fund Created.

Physical property costing $100 and having 10 years life. Annual net earnings $13.587, all paid out to stockholders; 6% considered as interest on investment, remainder as return of capital.

Year.	Investment at beginning of year.	Interest on investment.	Amount received by stockholders.	Investment reduced by:	Investment at end of year.
1	$100.00	$6.00	$13.587	$7.59	$92.41
2	92.41	5.55	13.587	8.04	84.37
3	84.37	5.07	13.587	8.52	75.85
4	75.85	4.55	13.587	9.04	66.81
5	66.81	4.01	13.587	9.58	57.23
6	57.23	3.44	13.587	10.15	47.08
7	47.08	2.83	13.587	10.76	36.32
8	36.32	2.18	13.587	11.41	24.91
9	24.91	1.50	13.587	12.09	12.82
10	12.82	0.77	13.587	12.82	0.00

Although Tables 25 and 26 show clearly enough that the same amount is required to meet the fair claims of the stockholders, whether or not a depreciation fund is established, as a matter of policy such a fund should always be created by companies operating under conditions in any way similar to those assumed, as this is the only way in which such a company can be reasonably sure of the continuity of its enterprise and avoid the use of antiquated equipment. Mr. Nicolaysen.

The second imaginary company to be investigated is assumed to begin operations under exactly the same conditions as the company previously described, the difference between the two being that the first operates one boat only, and the second, through the opening of new routes, increase of business, etc., is enabled to add one new boat to its equipment at the end of each of the first 9 years, so that it begins its second year with two boats, of which one is new and the other 1 year old; its third year with three boats of which one is new, one is 1 year old and one is 2 years old, and at the beginning of the tenth year, it operates ten boats, one just acquired and good for 10 years' service, the next, 1 year old, and able to serve 9 years more, and so on down to the original boat which has given service for 9 years and has a remaining useful life of only 1 year.

Each of these boats, as in the case of the first company, is assumed to cost \$100, to have a useful life of 10 years, and to earn, during every year of its operation, the sum of \$13.587 net, available for depreciation and dividend payments.

It is further assumed that the limit of the profitable expansion is reached with the tenth year, and that, from then on, the number of boats operated remains stationary at ten, one new boat being purchased annually to replace the oldest boat in service, so that the equipment operated during the eleventh year, twelfth year, etc., will correspond exactly to that operated during the tenth year.

It has been shown in the case of the company operating one boat, that net operating earnings of \$13.587 was sufficient to satisfy the fair claims of the stockholders, as it allowed the company to pay 6% dividends and create a depreciation fund which, at the end of 10 years, when the boat must be replaced, amounts to \$100, the original investment.

Now it is evident that the fair return per boat is independent of the number of boats operated, so that ten times \$13.587, or \$135.87, must be a fair return for a company operating ten boats, and provides both for depreciation and dividend payments. In the case of the company now under consideration, the condition of the equipment in the tenth year is maintained indefinitely by the annual replacement of one boat at a cost of \$100. This shows that the annual depreciation is \$100 and that the portion of earnings from operation available

Mr. Nicolaysen. for dividends is only $35.87, when the replacements are paid out of earnings. As the total "value new" of the equipment is $1 000, the interest on "value new", at 6%, is $60 in place of the $35.87 available for dividends; the latter amount is 6% of $597.80, which is thus the depreciated or present value on which interest should be figured.

The fact that earnings from operation are insufficient to provide both for the renewals of worn out boats and for a return at the assumed rate on "value new" of the physical property, does not prove that any portion of this value has been confiscated. It simply means that the total earnings include the return on the depreciation fund, created during the years when no renewals were required, and the combined earnings from operation and the depreciation fund—as shown in Table 27—will be found to be large enough to meet renewal charges and to yield the assumed return on the full "value new" of the equipment.

TABLE 27.—SINKING-FUND DEPRECIATION. DEPRECIATION FUND CREATED.

Physical property consists of boats costing $100 each and having 10 years life. One new boat purchased at the end of each year. Ten boats owned in the tenth and following years. Annual net earnings, $13.587 per boat operated. Stockholders pay for the first ten boats and receive 6% dividends. Remainder of earnings placed in depreciation fund that earns 6% per annum and pays the cost of renewals.

Year.	Capital invested.	Net operating earnings.	Return to stockholders.	Depreciation Fund.			
				Payment at end of year.	Interest during year.	Paid out for renewals.	Value at end of year.
1	$100.00	$13.587	$6.00	$7.587	$0.00	$0.00	$7.59
2	200.00	27.17	12.00	15.17	0.45	0.00	23.21
3	300.00	40.76	18.00	22.76	1.39	0.00	47.36
4	400.00	54.35	24.00	30.35	2.84	0.00	80.55
5	500.00	67.94	30.00	37.94	4.83	0.00	123.32
6	600.00	81.52	36.00	45.52	7.40	0.00	176.24
7	700.00	95.11	42.00	53.11	10.57	0.00	239.92
8	800.00	108.70	48.00	60.70	14.40	0.00	315.02
9	900.00	122.28	54.00	68.28	18.90	0.00	402.20
10	1 000.00	135.87	60.00	75.87	24.13	100.00	402.20
11	1 000.00	135.87	60.00	75.87	24.13	100.00	402.20
12	1 000.00	135.87	60.00	75.87	24.13	100.00	402.20

If no depreciation fund has been established during the development period, so that the company has no income other than that obtained from operation, then, of course, it is impossible to pay dividends at the assumed rate on more than the depreciated value, but the depreciation fund, in that case, is really invested in the equip-

ment, and the stockholders' equity in the same is, and has always been, equal to its depreciated value, a fact that is clearly shown in Table 28. Mr. Nicolaysen.

TABLE 28.—Sinking-Fund Depreciation. No Depreciation Fund Created.

Physical property consists of boats costing $100 each, and having 10 years life. One new boat purchased at the end of each year. Ten boats owned in the tenth and following years. Annual net earnings, $13.587 per boat operated. Stockholders pay in new capital as required and receive 6% dividends. Remainder of earnings invested in new equipment.

Year.	Stock holders' investment at beginning of year.	Net Operating Earnings.			Additional investment by stock-holders.	Stock-holders' investment at end of year.
		Total.	Return to stock-holders.	Invested in equipment.		
1	$100.00	$13.587	$6.00	$7.59	$92.41	$192.41
2	192.41	27.17	11.55	15.62	84.38	276.79
3	276.79	40.76	16.61	24.15	75.85	352.64
4	352.64	54.35	21.16	33.19	66.81	419.45
5	419.45	67.94	25.17	42.77	57.23	476.68
6	476.68	81.52	28.60	52.92	47.08	523.76
7	523.76	95.11	31.43	63.68	36.32	560.08
8	560.08	108.70	33.60	75.10	24.90	584.98
9	584.98	122.28	35.10	87.18	12.82	597.80
10	597.80	135.87	35.87	100.00	0.00	597.80
11	597.80	135.87	35.87	100.00	0.00	597.80
12	597.80	135.87	35.87	100.00	0.00	597.80

It is true that all the current earnings may have been paid out to the stockholders, and these may have paid in annually the full amount needed for additions, but this does not increase their investment any more than it would be decreased if all earnings should have been used for additions and no dividends paid, in which case the capital directly supplied by the stockholders falls short of their actual new investment by an amount equal to a fair return on the money previously invested.

In praxis, it will probably be found, in a majority of cases, that earnings during the first few years of a company's existence, are insufficient to provide for proper depreciation and dividend payments. In such cases the accumulated difference between what may be termed "fair earnings" and actual earnings, should normally be considered as development expenses and included in the intangible values. When this is done, it becomes unnecessary to assume as before that earnings from the very beginning shall be large enough to provide for a fair return on the value of the property. If early earnings are less than fair earnings, the company under consideration will later be entitled

Mr. Nicolaysen. to a return both on its physical property and its development expenses, but that portion of the return which is based on the physical property will be exactly the same as that previously found.

The assumed rates of return on capital invested and on the depreciation fund were made identical, for the reason that normally it will be found to be both possible and desirable to invest the depreciation fund in additions and betterments. In view of the fact that the element of risk is largely eliminated from regulated utilities, the speaker is inclined to consider the assumed rate—6%—as fair, but it is not his intention to claim that it is necessarily the proper one, the object being primarily to fix a certain rate so that the tables could be calculated.

The claim is often made that if a depreciated plant is able to give as satisfactory service to the public as it did when new, it should also give the same return to the owners. This argument is entirely beyond the speaker's comprehension. If its correctness was recognized, it would seem that the ferry company previously considered could be made a very profitable investment by the simple expedient of buying old, but still serviceable, boats in place of new ones.

The speaker believes that the apparently very great differences between the companies here considered, and actually existing utilities, on investigation, will be found to be unessential.

In all cases the physical property may be divided into three classes: Class I is to include all items of property renewed out of capital; Class II comprises all groups of items which through annual renewals out of earnings are maintained indefinitely in approximately their present condition; and Class III contains all groups of items which eventually will be included in Class II, but which have not yet reached the stage where renewals make up for depreciation. Justice is done, both to the public and to the stockholders, when earnings are large enough to provide for depreciation payments on the sinking-fund basis on all property in Classes I and III, and to yield in addition a fair return to the stockholders on intangible values, on the "value new" of property in Classes I and III and on the depreciated value of property in Class II.

Mr. Lippincott. J. B. Lippincott, M. Am. Soc. C. E. (by letter).—The report of this Committee is a most satisfactory and clear-cut discussion of this important subject. If the American Society of Civil Engineers can assist in standardizing methods for estimating reasonable rates for utilities in cities, it will be conferring a benefit both to the investor and to the public. The Committee states (page 21):

"The allowance for overhead charges has generally been substantially under-estimated by commissions and the Courts, as well as by engineers of limited practical experience in construction work. The

latter are prone to assume that unit prices bid by contractors or determined during the progress of the work by the inspector are fair measures of its final cost to the owner, but such is rarely the case." Mr. Lippincott.

In building an engineering work there are two classes of expenses: First, the expense of building the structure itself; and, second, the indirect expenses which are unavoidable and for which there is little to show after the work has been completed. Engineers who have been connected with large enterprises, and have carefully kept segregated cost accounts, are fully aware of the gravity of these indirect charges, whereas others, who have not had this experience, are apt to minimize them. It is to be regretted that so few such cost records are available. Estimates are prone to be below the actual cost. This is not due so much to failure to comprehend the cost of the structure itself, as to lack of realization of these unavoidable auxiliary, incidental, or general expenses. For the same reason, engineers or Courts who are evaluating properties are apt to consider only such structures as they may see on the ground. The worn-out tools and equipment, the monies paid for injuries to men and animals, or police protection, all have left on the ground no visible trace that may be appraised, yet they are clearly an expense incident to the construction.

Any official body whose duty it is to fix a rate for a public utility has a grave responsibility, not only to the public, but also to the investor. Justice requires that each shall be treated with fairness, and, usually, this is the intent. It is necessary for the proper development of a new country that great enterprises should be encouraged. Therefore, particularly if business profits, commensurate with the risk incurred, are not to be allowed in fixing the rates for utilities, and these institutions are not safeguarded against losses, all legitimate expenses incident to the construction should be fully included in the cost of the works, if this cost is to be the basis for fixing rates.

In the construction of the Los Angeles Aqueduct it was possible, to an unusual extent, to learn the real cost of the different classes of a great variety of work, as this structure was built on a day-labor basis instead of by contract, and because an accurate system of cost accounting was maintained. The writer was the Assistant Chief Engineer through its entire survey and construction. The engineer who directed and estimated on the work was in a position to follow closely the reasonableness of all expenses through these cost records. The Aqueduct was finished within the estimated cost; the organization was entirely free from politics, and the monies were all expended without scandal on account of waste or graft.

The methods of accounting used on the Harriman System of railroads were used on the Aqueduct; these provide for all work to be authorized by "engineers' work orders". Every item of expense was

Mr. Lippincott.

charged to some specific "work order". Labor distribution sheets and foreman's material diaries were used. The distribution of accounts was made in the field and checked at headquarters, both by a "Cost Engineer" and by the Auditing Department. The total expenditure and the work order charges were balanced monthly.

Mr. O. E. Clemens, an engineer who was engaged in the field during the early portion of the construction work, showed unusual adaptability to the matter of cost-keeping, and, for this reason, he was transferred to the office of the Chief Engineer and given the title of Cost Engineer, in which position he served during the greater portion of the period in which the Aqueduct was under construction. Subsequent to the completion of the Aqueduct, he made the final general summary and analysis of cost data from which the figures given in this discussion were obtained. The system, as developed by Mr. Clemens, was unusually satisfactory and efficient.

The expenditures were divided between the construction of (1) The Waterway and Its Appurtenances; (2) Auxiliary Construction Expenses and Operation; (3) General Miscellaneous Expense and Operation; and (4) General Office and Executive Expense.

"The Waterway and Its Appurtenances" (1) included the finished structure which is left as a result of the effort. This is mainly the conduit which conveys the water.

"The Auxiliary Construction Expense and Operation" (2) included surveys and general engineering of a preliminary nature, water-pipe lines, for construction and domestic use, telephones, roads and trails, buildings, low-tension power lines, division administration (which included superintendents, division engineers, and office force, material handling, sanitation, and housing), miscellaneous tests, expended cement sacks, patrol of the Aqueduct, miscellaneous losses, reorganization expense, concrete replacements, and net equipment expense after its salvage. This auxiliary expense is the largest indirect charge of this nature, and, for that reason, it may be desirable to give some further analysis of this account. It is divided as shown in Table 29.

The "General Miscellaneous Expense and Operation" (3) included passenger transportation for labor, water investigations, sundry adjustments, subsistence losses, unadjusted freights, stock service losses, and miscellaneous operation adjustments. This account was credited with revenues from the sale of domestic water during construction, rentals, and other incidental revenues.

The "General Office and Executive" (4) account included the salaries of all general officers, accounting, disbursing, purchasing department, and legal expenses, executive railroad transportation, and automobile service. Interest charges or development expenses are not included in any of these items.

Mr. Lippincott.

TABLE 29.—AUXILIARY CONSTRUCTION EXPENSES AND OPERATION (2).

Items.	Total cost.	Percentage of waterway cost.	Average cost per foot.
Surveys and general engineering	$366 932	2.30	$0.311
Pipe lines, net	281 645	2.77	0.375
" " M. and O.	162 780		
Telephones, net	75 638	0.95	0.127
" M. and O.	75 115		
Roads and trails	274 395	1.93	0.261
" " " M. and O.	34 237		
Building expense, net	336 531	2.10	0.285
Low-tension power lines	59 240	0.35	0.05
Division administration	775 412	4.86	0.657
Miscellaneous tests	39 167	0.25	0.033
Expended sacks	49 553	0.31	0.042
Patrol of Los Angeles Aqueduct	2 672		
Miscellaneous losses (not including M. O. H., final)	88 130	1.20	0.163
Reorganization	28 580		
Concrete replacements	72 752		
Equipment expense, net	1 548 721	9.20	1.31
Totals	$4 271 500	26.72	$3.614

The summary of costs of the aqueduct construction, exclusive of land purchases and cement plants, and the percentages as compared with cost of waterway, were as follows:

(1) Waterway	$15 942 489	....
(2) Auxiliary expense	4 271 500	26.72%
(3) Miscellaneous expense	277 038	1.75%
(4) General executive	843 944	5.30%
Total	$21 334 971	

Of this amount 33.77% of the waterway cost was for indirect charges. It may appear at first that these indirect charges were unusually high on this Aqueduct because of the remoteness of the work. However, transportation expenses, whether by rail or wagon, were included in the direct cost of the work. The auxiliary charges were practically as great on the end of the Aqueduct next to the City of Los Angeles as they were on the remote portions of the line.

In a large undertaking, these overhead expenses are proportionately less than on a smaller one. This general average was borne out consistently on the different divisions of the Aqueduct. On those divisions where the total construction costs were low, the percentage of overhead was relatively high, and where the total construction costs were high, the overhead charges were proportionately low. On the Los Angeles Aqueduct, during the greater portion of the work, no passenger

Mr. Lippincott. transportation was paid for labor. As the construction neared completion, however, and the length of time left for finishing the work was becoming short, some passenger transportation for labor had to be paid. The subsistence losses were largely for meals furnished men who never worked any time. These items are included under "Miscellaneous Expense" (3).

The first issue of bonds for the Los Angeles Aqueduct was for $1 500 000, in 1905. This issue bore interest at 4 per cent. This fund was used for the purchase of lands and water rights and for surveys and general investigations. It was followed, in 1907, by an election authorizing the issuance of $23 000 000 in bonds for construction purposes. Of this latter issue, $1 033 600 bore interest at 4% and the remainder at 4½ per cent. The total premium was $77 010, and the bonds were sold as required for funds to carry on the work. The Aqueduct was completed on schedule time in the summer of 1913, and the interest charges on all bonds up to that date (June 1st, 1913) amounted to $3 662 828.43, or 23% of the cost of the waterway. This does not include or take into account the main power-plant construction or interest thereon. This 23% added to the 33.77% for auxiliary expenses actually incurred on the Aqueduct, as previously stated, makes 56.77% total expense for these charges as compared with the actual waterway cost.

On work of a similar character carried on in a less remote region, certain adjustments of these percentages for auxiliary expenses may be made. Roads and trails, buildings and telephones, amounting to 4.98% on the Aqueduct, may be entirely eliminated. The freight on the equipment was a considerable portion of the charges against this item, and the 9.20% may reasonably be reduced to 8 per cent. Other expenses, for replacements, supervision, patrol, etc., should remain the same. We have then for the adjusted Aqueduct auxiliary expenses 26.72% — 6.18% = 20.54 per cent. In the case of work done in a highly developed region, there are more damage claims than on work in a more remote region. The miscellaneous and general executive expenses should not be affected materially. An ordinary work would not involve so long a period of construction, and, consequently, the interest charges would not be so high. However, if consideration be given to the period during which a water-works is being put into full use and earning power, an interest charge of 10% would not be unusual.

Due to the severe employers' liability law that has been passed recently in California, the heavy damages that may be collected by workmen who are injured, irrespective of whether or not they contribute to the accident by their own negligence, introduces an additional new expense in construction work in this State. The ordinary rate for insuring a pay-roll prior to the passing of this new law, against such damage claims, was 1 per cent. The insurance rates at

this time amount to \$5.25 per \$100.00 of the pay-roll for laying water mains, \$6.12 for erection of pumping stations, dams, and reservoirs, and \$12.25 for tunnel work. Owing to the magnitude of the Aqueduct construction, the Board of Public Works did not carry accident insurance, but settled its own damage claims, charging the losses to "Auxiliary Expense and Operation" account. This loss amounted to 0.26% of the pay-roll. However, this work was performed prior to the time when the new employers' liability law went into effect, and settlements were not made under its provisions. Allowing for the present insurance rates for water-works construction, there should be added to the auxiliary expenses for the above, 5% of the labor cost on the work. These labor costs on the Aqueduct were 60% of all costs, and probably amount to at least one-third of the total construction cost on an ordinary water-works. Consequently, 1.67% should be added for liability insurance in California. From this should be deducted the 0.26% already charged to this expense on the Los Angeles Aqueduct, or, say, 1.40% for a net amount. Mr. Lippincott.

All the figures in the estimate of cost of this Aqueduct are based on day labor, without profit. A board of four engineers, of which the writer was a member, was employed by the City to determine the value of the Los Angeles Water-Works which the City was purchasing. In this appraisal it was conceded that allowance should be made for contractor's profit, on the assumption that if a new plant was to be built quickly and without any basic organization, to replace the existing plant, the work would be done by contract and not by day labor, and that a reasonable allowance for contractor's profit would be 6 per cent. The Board states: "After due deliberation it is our firm conviction that a further 6% should be added for a (contractor's) profit." The arbitrators slightly increased these estimates, and the final amount paid was higher. On the cost of the completed structure, 6% would be a low allowance for contractor's profit on the construction of a water-works system.

Therefore, the following estimates of auxiliary cost for work similar to that done on the Los Angeles Aqueduct, adjusted to less remote localities, done by contract, and with 10% allowed for interest during construction, are given:

(2) Auxiliary expenses	20.54	per cent.
(3) Miscellaneous expense	1.75	"
(4) General executive	5.30	"
Interest during construction	10.00	"
Liability insurance	1.40	"
Contractor's profit	6.00	"
Total auxiliary expense	44.99	per cent.

Mr. Lippincott.

These figures are believed to justify an addition of 45% to the direct construction costs of a water-works of the general class previously referred to, to provide for auxiliary expenses.

In addition, a water-works is usually constructed in a fragmentary or piecemeal manner, and not as one comprehensive undertaking, all of which could be put through in a minimum period of time. Work done in small units is more expensive than if built as a whole, vigorously prosecuted, with ample financial backing, as is ordinarily assumed to be the case in making a valuation report. Cheap work must be done with the maximum organization that can be placed on the entire job efficiently, in order to reduce overhead expense. This argument would justify using higher unit cost figures, in making a valuation, than those which are available from a large piece of work that is built under full headway.

The writer believes that we have reached the time when the investors in many of the securities of public utilities need helpful support in the matter of reasonable rate fixing. Irrespective of indiscretions and bad management in the past, we should look forward to reasonable service and fair treatment in the future. If these institutions are not to be guaranteed against loss, and are only to be permitted to earn such reasonable interest on their investment as could be obtained from first-class mortgages or established commercial enterprises, then certainly, in determining the value of the property, due weight should be placed on all legitimate expenditures, including auxiliary costs. If this is not done, it will follow that either the service will deteriorate or the investors will refuse to continue to place funds in institutions of this nature. In the Southwest there is a decided movement on the part of utility corporations to dispose of their properties, even at a sacrifice, in order to terminate the contentious strife with the public. The California Railroad Commission expresses a realization of this situation in its findings in the Palo Alto Gas case:

"This brings me to a consideration of the final question in the case, namely, the amount of return to be allowed the Gas Company on its plant. No fixed percentage applicable to all cases and all classes of utilities can be established by this Commission. Each case must be judged on its own merit. It may be well that a utility in one community would be entitled to one rate of return while a similar concern in another community would be entitled to a different rate. It may be that a large and solidly established utility will not be entitled to as high a return as a smaller utility which is struggling against adverse circumstances. The most that can be said by way of general principles is that the return should be at least the average return which is earned by other classes of business of the same degree of hazard in the same community. The Commission in fixing a rate of return must be liberal, lest too strict a policy result in turning capital to other fields of enterprise. California needs development by public utilities, and this

Commission's policy should be a broad and liberal one, so as to encourage capital to develop the State by legitimate public utility enterprises where needed. The Commission should be careful not to permit an inflation of prices in ascertaining the value of the property of a public utility used and useful for the public purpose, but should be liberal in establishing the rate of return on that value. Bearing in mind all the facts of this case as shown by evidence, I find that a rate of return of 8 per cent. on the value of the property of the Palo Alto Gas Company used and useful for the puplic purpose, as fixed herein, is at least a fair and equitable rate of return. If anything, the rate is too high by reason of the fact that the Commission has been more than liberal in establishing the basis of value."* Mr. Lippincott.

In certain rate-fixing litigation before the California State Railroad Commission at the City of San Diego, in which that city was made a defendant in the complaint that originated from certain consumers outside of the city who were served from the municipal system, the Engineering Department of the Railroad Commission in its estimates of the value of the property, made the following allowances for overhead or auxiliary charges for the construction of the Otay and Moreno Dams and the Delzura Aqueduct, the total construction amounting to $2 900 000:

Engineering	5	per cent.
Contingencies	7	"
Interest for 3 years	9	"
Legal and general expense	2	"
Liability insurance	3	"
Total	26	per cent.

The writer is informed that these are the highest overhead charges that have been allowed for work of this character by the Engineering Department of the California Railroad Commission. It is fully realized that exception may be taken by some to the classification of the accounts on the Los Angeles Aqueduct, and the relatively high auxiliary expenses shown by this method of classification, but it is believed that the items are given in such detail that others may reclassify them in such a manner as to compare them with their own expenditures and with other distributions that have been made on other work.

Alfred Noble, Past-President, Am. Soc. C. E.† (by letter).—The writer believes that the subject of valuation is of such great importance that discussion should be continued at future meetings of the Society. Mr. Noble.

* Opinions and Orders of the Railroad Commission of California, City of Palo Alto *vs.* Palo Alto Gas Co., Vol. 2, pp. 316-317; Opinion of Thelen, Commissioner.

† This discussion was forwarded by Mr. Noble a short time before his death, and, consequently, could not be revised by him.

Mr. Noble. As one of the Committee responsible for this report, he desires to state that it represents the unanimous view of the six members of the Committee participating in its work, after two years of study. It does not follow that its conclusions are correct, and it is hoped that full discussion will disclose errors, if they exist.

The most general objection to the report is in regard to the treatment of depreciation. The Committee's plan provides for income through rates to return to the investor his capital in installments, corresponding with the estimated reduction of value through depreciation. If the investor receives, through rates, a return of the full investment, together with reimbursement of operating, maintenance, and other necessary expenses, and a due allowance for use of capital, risks, and management, it is claimed that he is justly treated and has no further claim on the public. If, instead of diverting to other uses the installments received on account of investment, he reinvests in extensions or replacements, on which fair earnings are allowed, he is receiving fair treatment from the public. If the Committee's plan does not accomplish this, it is faulty. It seems to the writer that the criticisms are in greater part due to misunderstanding of the Committee's recommendations. The Committee regards it as improbable that a large sinking fund will be maintained under any circumstances, because the owner of the property can make the money earn more by other investments; the preferable investment, where practicable, would be for extensions of the plant, if the money were not needed at the time for replacements.

The Equal-Annual-Payment Method has been proposed in order to make the collections from the public as nearly uniform as practicable during the life of the property, the increasing payments on account of return of investment balancing, or nearly balancing, the decreasing payments for use of capital. This is entirely consistent with the idea of a uniform rate for the same service, whether the items of the plant are new or old.

The return to the investor by the Equal-Annual-Payment Method differs from the Sinking-Fund Method, as usually understood, in this, that the sum of the annual returns of capital equals the investment without the inclusion of interest.

The writer's object in presenting this brief discussion is to ask careful examination of the Committee's plan, and to express not only confidence that it is based on justice to all, but a willingness to modify or abandon it, if it can be shown that this confidence is unwarranted.

Mr. Byers. M. L. Byers, M. Am. Soc. C. E.—After considering what has been said on this subject, it seems to the speaker that perhaps the time is opportune to endeavor to emphasize one or two points, and perhaps to ask the Committee for a little further explanation of the practical

application of its theories of valuation for rate-making, having in mind that it is stated (page 6), that "the valuation is for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more." Mr. Byers.

I.—The Courts state, and it seems equitable to assume, that, in the regulation of rates, the property is entitled to a fair return on its fair value.

$$\begin{aligned}\text{Fair Return} &= \text{Fair Value} \times \text{Fair Rate of Return}\\ &= \text{Output} \times \text{Tariff Rate (or Unit Selling Price)} - \text{Cost}\end{aligned}$$

or,

$$\text{Tariff Rate} = \frac{\text{Fair Value} \times \text{Fair Rate of Return} + \text{Cost}}{\text{Output}} \qquad (A)$$

As the tariff rate is something that is to apply to future earnings, apparently we must deal with fair value, and fair rate of return of the immediate future, as a practical proposition—what it would probably be for a year, or whatever period would elapse before the next time of revaluation, or readjustment of rates.

In Equation *A* fair value is determined through the process of valuation of the property. It being "the reasonable value of the property at the time it is being used for the public" which must be used, this varies with changes in the prices of labor and materials, land values, additions and betterments carried out, etc., etc.

Fair rate of return is a quantity which varies with the risk of the enterprise and with the value of money. The value of money is constantly fluctuating, and, therefore, for practical purposes, this must be based on the estimated average risk of the industry and the estimated average value of money during a reasonable period of the future. It cannot be determined accurately, but must be estimated by application of experience and judgment to the problem.

Cost is an item which must be estimated. It depends on output, and output will be greater or less in the railroad business, depending on the tariff rate. If the tariff rate is placed so high that it prevents the business from being handled, the output will be reduced, and the unit cost will naturally be increased because of the overhead charge. Cost also includes a number of items, such as, maintenance, cost of operation, etc., including depreciation.

Output is an item which, in the railroad business, depends on the condition of prosperity of the country, the state of the crops, the tariff rate which is adopted in a particular case, etc. If we have rainfall at the proper time, then the output will be increased. Aided by experience, these must be forecasted in determining and estimating output, which is to be applied in the determination of the tariff rates to be used in the future.

Mr. Byers. It is clear that, in applying this equation to a gas plant or other utility of that sort selling one commodity, it is comparatively easy to solve, even though the various factors seem to be not of the most definite. When it comes to applying it to a railroad, the speaker must confess that he would like the Committee to go into a little detail as to the practical application of its theories, for now Equation (*A*) becomes:

$$\underset{\text{(thousands of them)}}{\text{Tariff Rates}} = \frac{\text{Fair Value} \times \text{Fair Rate of Return} + \text{Cost}}{\underset{\text{(thousands of them)}}{\text{Outputs}}} \ldots (B)$$

There is still only one equation, but there now become thousands of unknown quantities.

Consider a system like the Pennsylvania Railroad, or the New York Central, or the Southern Railway, or any of the large railway systems. There are hundreds and thousands of stations, there are dozens of terminals, each one of which means a different amount of plant applied for the purpose of transportation.

It is common knowledge that, in the passenger business, passengers are carried over quite a number of lines, and are carried various distances. Under present practices, one passenger is charged one rate to-day because he buys a single-trip ticket; another passenger pays a little different rate because he buys a return ticket; still another passenger going between the same points is charged a little different rate because he buys a fifty-trip ticket; still another passenger has a mileage book, and he pays a different rate. Now, one important thing to emphasize in this particular is this: that these passengers have built their homes, many of them at different places, on different lines, because they could travel at those particular rates. They have their investments made.

The freight situation is the same throughout the whole country. Plants all over the United States have been located where they are, because, under the rate system now in existence, those were the economical places to locate those particular plants. Those rates have not been determined on any such basis as this. They have been determined through the influence of many factors.

If it is attempted to substitute for this system of rates which has been developing in this country for 50 years, a new system built on an entirely different basis, what will be the results to the industries which have been economically located on the entirely different present basis? This point is raised simply to emphasize the importance of this question and the danger of throwing the weight of this Society's influence for or against this proposition without due consideration.

It not only affects the railroads' interests, but millions and millions of capital invested in every form of industry.

Mr. Byers.

The bituminous coal traffic furnishes an interesting point of view from which to consider the use of "Valuation for the Purpose of Rate-Making." An enormous quantity of bituminous coal is carried by various railroads to the Great Lakes for transportation across these lakes to points of consumption beyond. Many coal fields, such as the Brazil, the Ohio No. 2, the Ironton, the Pocahontas, the Kanawha, the Fairmont, the Pittsburgh, the Connellsville, etc., supply the coal for this traffic. These coals compete with each other in the Great Lakes markets, commanding prices at such common terminal points, which depend on the relative qualities of the coals and on no other factors. For example: Pittsburgh coal, due to its better quality as a gas coal, commands a price about 10 cents per ton higher than Ohio No. 2 coal. The cost of mining these different coals per ton varies greatly, depending on the thickness of the seam, character of the roof, equipment of the mine, daily output of the mine, wages of the miners, etc. The distance of the coal from the common markets varies by hundreds of miles by air line, and by railroad routes over which it is transported. The cost of transportation to the railroad company varies with the distance, grades, etc. Under the competitive conditions of to-day and of the past the wages of the miners in the different fields, the freight rate per ton (not per ton-mile), and the selling price of the coal at the mines, have been worked out so that the net profits of the industry (market price minus actual cost of mining, transportation, etc.) are divided between the different parties at interest, namely, the miner, the mine owner, the railroad, etc., on the basis which such free competitive condition has established to be equitable.

May it now be asked of the Committee how, in its opinion, it is possible, bearing in mind such complication of service rendered by the railroad, to utilize "Valuation for the Purpose of Rate-Making" and if, in its opinion, under the peculiar methods and conditions which it would seem must be used (if it can be done at all) in the application of valuation to this purpose, it has included in its report all the factors which must enter into a valuation to be used under such unusual circumstances? To one who has had experience in railway operation, but who has not had experience in the facility with which valuation is applied (no doubt) to the simpler provinces of rate-making for gas and similar utilities, it would be a material source of satisfaction if the Committee could feel it to be a part of its duty in the making of a report on "Valuation for the Purpose of Rate-Making" for a railroad, to indicate how such valuation is to be practically applied.

Assume, for example, that valuation is to be applied for the purpose of rate-making to the railroad which hauls Ohio No. 2 coal (as well as other products) to the Lakes, there to meet the competition

Mr. Byers.

from the other coalfields transporting their products over other railroads. In order to determine tariff rate, it is necessary, among other things, to estimate both cost of operation and output; but the tariff rate on the Ohio No. 2 coal, which is arrived at as the result of the application of this valuation data, may be so high as to prevent entirely the movement of this coal; or the rates established through valuation on other lines may prevent the movement of the other coals and give the entire market to the Ohio No. 2 coal. There is nothing said, in the Committee's list of principles, about the necessity for first valuing and establishing rates on all competing lines before attempting to value and establish rates on the line which is the immediate subject of the inquiry. Yet, how else could the Committee be enabled to compute either cost or output for the line transporting Ohio No. 2 coal, and, in the absence of these figures, how could the valuation be used "for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more?"

It would seem that the Committee was well within the bounds of fact when it stated (page 67): "There are several reasons why a physical valuation, taken by itself, furnishes an unsatisfactory basis for determining rates."

II.—In three ways, the Committee has made the discussion of its report unusually difficult.

In the first place, though the application of its recommendations to railroad properties is, at the present time, probably of the maximum interest, practically all its illustrations and applications of principles are to other public service corporations, differing very much in character from railways.

In the second place, although the Committee is dealing with a subject about which there is a very great amount of misapprehension and looseness of use of terms, it does not furnish definitions of the terms which it uses, and these terms, as applied to railway properties, become, many of them, extremely uncertain in their meaning.

Lastly, it frequently states its view without explanation or proof of its correctness, as where it states, as one of the general principles involved in valuation for rate-making (page 6): "It has been well settled by the highest Courts that the owner of such property is entitled to a fair return upon the fair value of the property utilized in or reasonably necessary to the service," and omits the important additional provision that it shall be a "fair return upon the reasonable value of the property at the time it is being used for the public." Another example: On page 67, the statement is made, "Embankments and the slopes of cuts may become more stable on account of the growth of grass and weeds, and may therefore be considered to be more valuable, but such value as this, which is not the result of

expenditure by the corporation, should have no place in the valuation of the property." Why not, as it represents value at the time of public use? Mr. Byers.

On page 69, it is stated:

"It has been held by the Committee that the unfavorably located public service property is not entitled to more than reasonable rates for the service it renders even though the property in question becomes a losing venture; conversely, the Committee believes that the favorably located older property should equitably receive liberal treatment by rate-making bodies."

Applied to railroad property, what does the Committee mean by "unfavorably located public service property" and by "unreasonable rates"? The New York Central and Hudson River Railroad follows the east bank of the Hudson River from Albany through Poughkeepsie to New York, whereas the West Shore Railroad follows the west bank from Albany through Newburgh to Hoboken, and *via* ferry to New York. Presumably, the Committee has in mind that, with reference to the Albany-New York business, the West Shore is an unfavorably located public service property as compared with the New York Central. Does it, however, consider the West Shore to be comparatively unfavorably located with reference to business handled between Albany and Newburgh, or between Newburgh and Hoboken, and would it call the New York Central an "unfavorably located public service property" because it cannot transport between these points as conveniently or economically as the West Shore? Probably without exception, every railroad serves some territories that are not served by any other railroad. How then, can any railroad be said to be an unfavorably located public service property, and if there is no such thing as an unfavorably located public service railroad property, then wherein does the quoted view of the Committee apply?

Old timers relate that, years ago, prior to the construction of certain narow-gauge railroads in the oil fields of Northwestern Pennsylvania, it was necessary to haul boilers and other apparatus for well-drilling plants many miles by teams of horses. When the railroads began to reach these developing territories, they adopted a freight rate for the transportation of boilers just a few dollars below the cost of transporting them by teams. Evidently, even this reduction in cost was a benefit to the shipper. Does the Committee consider such a rate as unreasonable under the circumstances, recalling that the traffic was light and high rates were necessary in order to pay a fair return on the cost of the railroad property? If the Committee does not regard this as an unreasonable basis of rates under the circumstances, does it not follow that, where necessary to earn a fair return on the fair value of a railroad property, any rate which

Mr. Byers. is low enough to permit the business to be moved by rail as against movement by other forms of transportation is a reasonable rate?

III.—The fundamental law governing the use of capital is that it will flow to the point where, risk considered, the prospect of remuneration for its use is the most enticing, and that it will flow away from any point where the prospect of remuneration, risk considered, is not satisfactory as compared with investment opportunities elsewhere.

Capital invested in fixed property, by reason of its immobility, is more vulnerable to injury than liquid capital. The latter, it has been found, after all sorts of efforts to control it, is absolutely free and can only be enticed, not commanded.

In estimating the risk of an investment in fixed property, capital is guided largely by previous experiences with such investments. If previous experience shows that it has been enticed into such investments by false pretenses and has afterward been robbed of the advantages promised it, the result will naturally be an appreciation of the item of risk and a consequent increase in the rate of remuneration demanded for this type of investment in the future.

If, for example, capital is enticed into the transportation industry by the promise of land grants, bonuses, etc., and, later, finds that efforts are made, whenever opportunity offers, to strip it of the fruits of such grants, gifts, etc., through direction or indirection, the result will be to make capital more wary of and to demand higher returns from investments in such industry in the future.

Always, when capital considers investment in any enterprise, it has two fundamental considerations before it:

A.—What of the conditions of return of this capital proposed to be invested?

B.—What of the remuneration for use during the period of investment?

Return of capital is made in two principal ways:

1.—Direct return, as when a mortgage matures and is paid off.

2.—Indirect return, as when the property is sold. In this class, the chance of appreciation or depreciation in sales value must be reckoned with.

The rate demanded by capital for its use is determined by two principal factors: the current value of money (depending on ratio of supply and demand) and the insurance rate demanded by the intensity of the risk.

It is not greatly material, a common understanding being assumed, how this total rate is made up, the principal point being the size of the total. For example: If it requires a return of 8% to entice capital into the transportation industry, it is immaterial to capital whether the

total amount represented by this 8% return is the result of a straight 8% return on the total amount of the capital invested; or a 10% return on 80% of such capital (the computer amusing himself with the idea that the 20% of capital eliminated is something which he terms depreciation); or a 6% rate on the original capital investment, plus an allowance equal to 2% on such capital, such allowance representing an 8% return on what the computer is pleased to term appreciation. The important point to realize is that, barring deception, unless the computer handles his computations so that the necessary total is reached, capital cannot be attracted into the enterprise. Mr. Byers.

On page 50, of the report, the Committee states:

"The present practice generally gives to the corporation, as to the individual, the so-called unearned increment due to appreciation in land values.

"This is incompatible with the fundamental principle that the corporation is entitled to fair return upon the fair value of its property, because, if the land is valued at the increased price, and the appreciation in value of the land is not included in the accounting, the corporation would receive a sum in addition to a fair return amounting to the appreciation in the value of the land."

On page 52, the Committee states in part:

"Where the valuation cannot be made in this way [that is, by taking the cost of recently purchased land], it is the view of the Committee that land can be valued most correctly on the basis of present price of neighboring land of similar character, augmented by the ratio ordinarily found to obtain in that region between land acquired by public service corporations on one hand, and by private parties on the other."

These two principles appear to conflict; for in the second quotation it is clearly intended that there shall be included in fair value the present value of the land, regardless of the fact that this present value is the result of past appreciation. Presumably, the Committee intends that the statement first quoted shall apply only to further appreciation of land after the date of the valuation. Does not the truth of this proposition, however, depend on the definition which the Committee intends to adopt for "fair rate of return", bearing in mind that it is not the method of computation of the fair rate of return, but the amount of such rate, which is essential in attracting or repelling capital? Most industries include land among the properties owned and used therein, and it seems reasonable to assume that the owners in each case expect to secure, in some form, whatever value accrues from the appreciation thereof. When it is said that a certain corporation earned 12% on its capitalization, this is, in the usual sense of the term, in addition to whatever appreciation in values may have occurred during such period. Is not the Committee injecting a very unusual meaning into its use of terms, and one

Mr. Byers. better calculated to confuse than to clarify the discussion? Certainly if, in other industries of equal risk, it is possible to earn a certain percentage on actual investment, augmented from time to time by land and other appreciation, then fair promise of similar earning must be extended to induce capital to enter the railroad industry.

It is often to advantage to use an extreme case for the purpose of illustration. Let it be assumed, therefore, that a certain corporation begins business with a capital of $1 000 000, of which practically nothing has been expended for land, although a considerable tract of land has been acquired and is used in the industry. The corporation earns, at the start, 10% on its capitalization of $1 000 000. Twenty years later everything with reference to the business and capital of the corporation is unchanged, except that the actual sales value of its land now amounts to $1 000 000. If the corporation went out of business, a new corporation would be obliged to invest $2 000 000 of capital in order to continue the business, and it certainly would expect to earn a fair rate of return—say, 10%—on such $2 000 000 capitalization. Is it not unjust, therefore, to require the old corporation to earn but 10% on the original outlay instead of on the present value? Moreover, how does the Committee reconcile its position in this matter with the statement of the Federal Supreme Court in San Diego Land and Town Company *v.* National City, also quoted with approval in the Minnesota Rate Case Decision, which reads as follows:

"What the company is entitled to demand in order that it may have just compensation is a fair return upon the reasonable value of the property at the time it is being used for the public."

Perhaps the Committee intended to convey the meaning that failure to allow unearned increment was intended simply as a practical measure existing only during short intervals and for the purpose of reducing clerical and similar difficulties. This would seem to be borne out by its statement on page 50, as follows:

"Your Committee therefore recommends * * * that the land or other property which appreciates in value be valued at present prices, and that the appreciation in the value of such property be offset against the depreciation in the value of the other property."

If such is the case, it is suggested that the situation be cleared up and that a number of definitions would be of material value in this connection.

IV.—On page 65, the Committee states:

"From the time that a public service commission is authorized to control rates, to fix the amount that the corporation may earn as a depreciation allowance, and to control methods of accounting, it is desirable and proper to take into account the excess or deficiency of earnings."

Does the Committee really consider that, the commission having authority to control rates, the fixing of the depreciation allowance and the control of the methods of accounting are items of consequence having bearing on the propriety of taking into account the excess or deficiency of earnings? Is not the control of rates the one essential factor? Is the Committee to be understood as stating that, in its opinion, in any valuation of the railways of the United States, "it is desirable and proper to take into account the excess or deficiency of earnings" since the appointment of the Interstate Commerce Commission with authority to control rates, and that prior to this date the insufficiency or excess of earnings of railways have "nothing to do with the case", as indicated by its statement (page 65): "It is the view of the Committee that an excess or deficiency of past earnings of an unregulated property should not be considered in making a valuation of the property," or does it take the view expressed (page 9) that, "in the case of a property recently created, with accounts kept properly, the best basis for a valuation is the actual reasonable cost as shown by the accounts," assuming that such a property could be found. Mr. Byers.

FREDERIC P. STEARNS, PAST-PRESIDENT, AM. SOC. C. E.—The Committee will not be able to close the discussion of its Progress Report on Valuation for the Purpose of Rate-Making before the latter part of the year, and the speaker thinks it desirable to reply sooner to certain features of the discussions printed in the February and March *Proceedings,* and of the oral discussions at the meetings of March 11th and April 2d, 1914. Mr. Stearns.

One of the most common criticisms of the report of the Committee is that it should not have reported on valuation for the purpose of rate-making.

The Committee has stated in its report that valuations for different purposes necessarily differ in some respects, and it took up first the one branch of the subject indicated by the title of the report. It is rather surprising that there is criticism of this proceeding expressed in so nearly the same terms by so many of those discussing the subject, because the matter has been previously discussed by the Society without such criticism.

For instance, in 1912, C. E. Grunsky, M. Am. Soc. C. E., presented a paper entitled "The Appraisal of Public Service Properties as a Basis for the Regulation of Rates".* In 1913, William J. Wilgus, M. Am. Soc. C. E., presented a valuable paper on the "Physical Valuation of Railroads",† which begins as follows:

"The Herculean task of valuing the railroads of the United States is of vital importance to the entire nation, whether viewed from

* *Transactions,* Am. Soc. C. E., Vol. LXXV, p. 770.

† *Transactions,* Am. Soc. C. E., Vol. LXXVII, p. 203.

Mr. Stearns. the standpoint of rate regulation or its possible outcome, government ownership."

At several places in this paper and in the discussions which followed, reference is made to valuation for the purpose of rate regulation, and to the difference between valuation for purchase and sale and valuation for rate regulation, it being claimed that value new should be used in connection with rate regulation.

Onward Bates, Past-President, Am. Soc. C. E., in the opening statement of his discussion* of the Committee's report, says:

"The valuation of public utilities for the purpose of rate-making is one of the great economic questions of the present day in the United States."

In the speaker's opinion, the physical valuation is the most important factor entering into the valuation of public utilities for the purpose of rate-making, and this Society includes the men who are best capable of making a physical valuation of such properties.

Mr. Robert H. Whitten, in his comprehensive work, "Valuation of Public Service Corporations", states:

"Official valuations of the property of public service corporations are made for four general purposes: (1) Taxation; (2) Accounting and capitalization; (3) Public purchase; (4) Rate-making."

And his treatise is devoted mainly to valuation for rate-making and public purchase.

The United States Supreme Court has consistently ruled for many years that public utilities are entitled to a fair return on the fair value of their property, and a very important element in the value of a property is its physical value.

A Federal Judge in California, who has had much to do with very important rate cases, stated that the most important fact to be determined is the value of the property.

Many of those who criticize the Committee for making a report on valuation for rate-making remind one of the defendant in the case of the borrowed kettle—first, that it was not borrowed; second, that it was cracked—and suggest that though valuation for the purpose of rate-making may be applicable to other utilities, it is not applicable to railways.

C. W. Hudson, M. Am. Soc. C. E.,† one of the critics of this feature of the Committee's report, goes so far as to hold that a railway is not a public utility, as he says it "appears advisable to the writer to separate the rules and methods for valuing public utility properties from those for valuing railway properties."

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* *Proceedings*, Am. Soc. C. E., for February, 1914, p. 341.

† *Proceedings*, Am. Soc. C. E., for March, 1914, p. 713.

These criticisms lead the speaker to ask, wherein is there a fundamental difference between the property of railroads and other public utilities, which affects the general principles to be applied in valuation for rate-making? Is not the railroad, as well as any other utility, entitled to a fair return on the fair value of its property? Is it not entitled to an annual allowance for depreciation to cover the lessening value of the various items of perishable property? If its value is to be determined, is it not entitled to have included the overhead charges and development expenses, as much as any other public utility, and is it not proper for this Society to discuss how all these subjects shall be treated to produce the most equitable results? Mr. Stearns.

There are many features, not taken up in the report, which need consideration, such as the effect of competition and of interstate conditions, and as Commissioner Maltbie stated, when discussing this subject, "there is no one yard-stick which will apply to all cases."

Every case is likely to present different conditions, and a public service commission charged with the duty of rate regulation will necessarily be confronted with special problems in different cases, which are likely to modify its action. The fundamental principles are the same in all cases.

If railroads are to be excluded because they are competitive and interstate, why should not all other competitive and interstate properties also be excluded, such as telegraphs, telephones, interurban street railways, pipe lines, expresses, etc.?

The speaker is pleased to note that the fundamental principles which the Committee has presented have not been seriously attacked in any direct manner, although they are ignored in much of the discussion. One of these principles is that the owner of the public utility is entitled to a fair return on the fair value of the property utilized in or reasonably necessary to the service. Another is that in any regulation of rates "there shall be taken into account two distinct features:

"First, the annual return, covering interest and profit, to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment.

"Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

Many have attacked indirectly the second of these general principles, holding that it is enough to earn and lay aside only the sums required for the renewal of plant units when they reach the end of their life. Such allowance as they would provide for depreciation, which they

Mr. Stearns. sometimes term "deferred renewals", is much less than the annual sum which the Committee believes the corporations are entitled to earn on account of depreciation.

In addition to the fundamental principle that the corporations are entitled to annual returns for the use of and waste of capital, it is obvious that they are entitled to be reimbursed for current expenses, including taxes, and that there are other factors entering into the determination of rates.

The Committee also regards it as fundamental that the rates should be made large enough to be attractive to capital, and expressed the view that "the public may be as injuriously affected by too low as by too high rates".

Several members have criticized the Committee on the ground that it has shown deference to the opinions of the Courts, and one member thought it did not show deference enough. There is no doubt that a Court is fallible, and that, in many matters, it is dependent on the testimony and arguments brought before it, but when an able Court, like the Supreme Court of the United States, composed of men who are able to take a broad view of matters which are presented to them, makes a decision to which it adheres consistently for many years, such decision should be deferred to, for two reasons: First, that it is probably a just decision; second, that it represents the highest law of the country, and a neglect to comply with it in cases coming before the Courts will result in failure.

The decision of the Supreme Court which is especially objected to is that relating to depreciation in the case of Knoxville *vs.* Knoxville Water Company, which, in the judgment of the speaker, is logical and just, and, within the speaker's knowledge, has produced higher returns for a corporation than it had been permitted by a rate-making body to receive prior to that decision.

The decision referred to was given in a rate case, and states clearly that "some substantial allowance for depreciation ought to have been made in this case" from the cost of reproduction. Four years later, in a railroad case, the same Court affirmed the same view by this statement: "and when an estimate of value is made on the basis of reproduction new, the extent of the existing depreciation should be shown and deducted". The decision also says: "It is also to be noted that the depreciation in question is not that which has been overcome by repairs and replacements, but is the actual, existing depreciation in the plant as compared with a new one".

It is better to defer to such well-established rulings of the Court, and to adopt methods of depreciation which will give equitable results under such rulings, than to adopt other methods which will be confiscatory if the Courts continue to rule in accordance with this well-established precedent.

Mr. Stearns.

It seems to the speaker that those so strenuously claiming value new and the smaller annual sum for depreciation which properly goes with it, while the Court decisions are as they are, occupy much the position of a man who wishes to walk to a destination beyond a high stone wall. There are two paths of equal length leading toward this destination: On one the opening has been closed, and on the other the opening is still available. The man would be successful in reaching his destination who goes by the unobstructed path.

C. B. Burdick, M. Am. Soc. C. E.,* apparently thinks that the Committee has not deferred sufficiently to the decisions of the Courts, and asks: "Has the Committee had the opportunity to consider the matter sufficiently to warrant it in recommending procedure at variance with the law as established by the higher Courts?" He adds: "The law is the outgrowth of the experience of centuries".

There are many matters connected with valuation on which the Courts have necessarily acted, but which cannot yet be regarded as well settled. Unprejudiced and intelligent discussion should aid in settling them correctly.

Several have discussed the question as to whether present or original physical conditions should be adopted in making an estimate of the cost of reproduction of an old property. It is recognized that changes are continually occurring, which greatly affect the cost of reproduction of a property, as, for instance, a pavement may be placed over a water pipe which was laid before there was a pavement; or a railroad may be built so as to give access to a dam which was originally built at a long distance from the railroad.

Owing to such changes as these, if present conditions are to be used, the cost of reproducing the water pipe would be largely increased by the cost of taking up and replacing the pavement, and the cost of the dam would be greatly decreased by reason of the diminished cost of providing all supplies and materials under the new conditions.

The Committee has taken the view that the unearned increment, or the unearned decrement, resulting from these changes is not required by any consideration of equity, and that when the estimated cost of reproduction is based on original physical conditions the most equitable results are obtained.

Of those who have discussed the subject, J. E. Gibson, M. Am. Soc. C. E.,† argues that the owner of a public utility is entitled to the unearned increment of value due to highway improvements constructed by the public, but he intimates that a decrement resulting from applying present conditions should not be treated in the same way, as he says:

* *Proceedings*, Am. Soc. C. E., for March, 1914, p. 706.

† *Proceedings*, Am. Soc. C. E., for February, 1914, p. 368.

Mr. Stearns. "The board of appraisers, in considering the present conditions of the plant, should also consider the conditions existing previous to the commencement of construction; otherwise, the utility might be done a great injustice."

Henry Floy, M. Am. Soc. C. E.,* believes in reproduction new under present conditions.

Allen Hazen, M. Am. Soc. C. E.,† apparently takes the same position as Mr. Floy, as he does not admit that the history of a water pipe "has anything to do with the cost of reproduction of the property as it now stands, or with its value".

Mr. Burdick shows the Committee's position in the matter of present or original conditions quite well when he says:

"It would seem that the Committee has shown a leaning toward investment as a measure of value for the purposes of rate-making, but through the influence of Court decisions, which are numerous and strong, had been forced to the adoption of present prices in the estimates of value".

The use of present prices prevailing at or near the time of valuation is made necessary by Court decisions, and also may be defended on the ground that it is the usual custom in valuing properties. The owner of the utility in this instance does not get an unearned increment or decrement, because he has to take the risk of both increase and decrease in prices. It is not desirable, however, that there should be changes in the valuation of property owing to changes in prices, and this can be avoided in the future with suitable legislation and with continuous commission control, as has been pointed out by the Committee.

In reference to the criticism that the Committee should be consistent and either use original prices with original physical conditions or present prices with present physical conditions, the speaker sees no inconsistency in the Committee's recommendation. The increments and decrements of value due to rise and fall in prices appear to be unavoidable under present conditions, but this does not furnish a reason for introducing increments and decrements due to changing physical conditions, especially as, in many cases, they would be very large.

Mr. Burdick says: "It would seem most logical in the valuation of these properties to pursue one logical method through to the end. If it be the cost of duplication at present, then reproduce under present circumstances", and yet, referring to the reproduction cost of a reservoir, he does not accept the present circumstances under which the railroad passes around the reservoir, but reverts to original conditions and says: "If a railroad formerly occupied the site of the reservoir,

* *Proceedings*, Am. Soc. C. E., for April, 1914, p. 1126.

† *Proceedings*, Am. Soc. C. E., for March, 1914, p. 713.

it would be entirely reasonable to assume that it would necessarily be removed in reproducing at the present time". Mr. Stearns.

The next point to be taken up is the question as to whether value new should be used in all cases where a valuation is made for the purpose of rate-making.

A very large number of those discussing the Committee's report contend that value new should be used in all cases. They do not discriminate, as the Committee has done, and state that value new should be used with some methods of providing for depreciation and not with others, but apparently claim that it should be used in all cases.

George F. Swain, Past-President, Am. Soc. C. E., has presented this view in the most extended and forceful manner, and his discussion will be the basis of the speaker's remarks on the subject. It may seem rather ungracious to select his discussion for criticism in this respect, because he has paid the Committee the high compliment of commenting favorably on all parts of its report except that relating to depreciation, and signifies his agreement with its conclusions as to most of the other features of the report.

In regard to the subject of depreciation, he believes "that the views of the Committee are lacking in clearness, that they involve confusion of thought and are misleading". A comparison of the views of the Committee and of Mr. Swain is, therefore, in order.

First, as to fundamental principles. the Committee, in its report (page 32), takes the view that:

"The corporation, under normal conditions, in addition to an annual return for the use of its capital, is entitled, as already stated, to an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value."

It also takes the view that the money received by the corporation for depreciation allowances should not be returned to the stockholders as a part of the income of the property, but should be retained as a part of the capital account. In portions of his discussion, Mr. Swain appears to be in entire accord with these views of the Committee. He says:

"The Committee is perfectly right in stating (Addendum, page 3): 'that the owner of a public utility property is entitled to have the rates made high enough, under normal conditions, to pay for the lessening value of the perishable property, so that when such property goes out of use he shall have received, not only a fair return for the use of the capital invested in such property, but shall also have the capital invested in it paid back to him,' except that the original capital should not be paid back to the investor, but should be held by the corporation for renewal when the time comes."

Mr. Stearns.

His exception does not represent a radical difference from the views of the Committee, as it fully agrees with his statement "that the original capital should not be paid back to the investor", and differs in regard to the last part of the exception only in the fact that the Committee would have the capital paid back to the owner of the property reinvested in the property when practicable, without regard to whether the reinvestment is in additions or renewals, and when this course is not practicable held temporarily in a reserve fund or used for retiring outstanding obligations.

In another place he makes this statement, regarding the fundamental principles of depreciation:

"In the speaker's opinion, the method of depreciation to be allowed is an accounting problem, the object being simply to set aside out of earnings each year such sum as may enable all renewals in kind to be made without any increase of capital."

After the endorsement of the statement made by the Committee, this is somewhat "confusing", as it limits the earnings on account of depreciation to the sum which will enable all renewals to be made without increase of capital, but makes no provision for obsolescence.

He refers to the depreciation allowance as being "a payment to the corporation and not to the stockholder who has invested his money", and adds: "It is a payment which should not reach the stockholder, but should be held by the corporation for a specific purpose." This is in entire accord with the views of the Committee.

After this declaration, it is somewhat "confusing" to find the statement that it is not of any consequence to the rate-payers, as such, "what the company does with the depreciation allowances which they have paid. It makes no difference to them whether the company puts these depreciation allowances in a fund * * * or whether it distributes them as dividends"; and then a later statement: "The rate-payer, of course, is concerned that the depreciation funds should be safely invested."

Many rate-payers have found to their sorrow that depreciation funds distributed to the stockholders as dividends are not "safely invested", and that the practice has resulted in unsatisfactory service and in demands for higher rates.

Mr. Swain claims that value new should be used with the Straight-Line or Equal-Annual-Payment Methods of depreciation, as well as with the Sinking-Fund Method.

The speaker will test this view by an example, using the item of property referred to in the Committee's tables on pages 34 and 40 of its report. The assumption is a property having a 20-year life, valued when new at $100, and with computations of depreciation allowances based on 5% interest compounded annually. If such a property is to

be valued new under different methods of depreciation, the annual allowance for depreciation by the Sinking-Fund Method would be $3.02, and by the Straight-Line Method, $5.00. Mr. Stearns

The Sinking-Fund Method used in connection with value new is, in the judgment of the Committee, equitable, as will be seen by reference to page 72 of its report, where it says: "*The Sinking-Fund Method* assumes that returns are to be based on the full value of an item of property." Mr. Swain also supports this view.

If the return for the use of capital is reckoned at 7%, the rate-payers, under the Sinking-Fund Method, would pay $7.00 + $3.02 = $10.02, each year of the life of this item of property for the use and amortization of the capital invested in it. Under the Straight-Line Method, if value new is used, they would be required to pay instead, for the same purposes, $12.00 each year. Is it not "confusing" to claim that $10.02 per year is equitable as a return for these purposes, and that $12.00 is also equitable? If the $10.02 is an equitable amount, as both Mr. Swain and the Committee agree, then the $12.00 represents each year a confiscation of the property of the rate-payers equal to practically 2% of the cost of this item of property.

The Straight-Line Method becomes equitable as between the rate-payer and the corporation only when the valuation of the item of property is reduced from year to year by the amount which the investment in that particular item of property has been repaid by the rate-payers. This does not mean that the investor in the original property will not continue to receive returns on the full amount of his investment, because the part of the investment which is returned to him through the depreciation allowances can be reinvested just as the investor in a note, when a part of the principal has been repaid to him, can reinvest the money so received and continue to receive interest on the full amount of his principal, although not from the holder of the original note.

The inequitable results obtained when using value new with the Straight-Line Method also obtain when using such value with the Equal-Annual-Payment Method, but the amount confiscated from the rate-payer in the latter case is not as large as when the comparison is made with the Straight-Line Method.

Mr. Swain, referring to Table 5 in the Addendum to the report, quotes the statement of the Committee that:

> "It would be grossly unjust to the rate-payers to permit the owner of the property, under the conditions presented, to earn dividends, not only on the $100 000 which he has invested, but also on the additional $117 609 contributed by the rate-payers, and yet this would be the result of using value new in connection with the Equal-Annual-Payment Method of providing for depreciation."

Mr. Stearns.

He then makes this statement:

"The speaker considers it entirely incorrect, with the understanding that the company must renew an item, when worn out, at its own expense without increase of capital. It is entitled to earn a fair return on the items in the last column plus an original depreciation allowance, and at the end of the tenth year, under these circumstances, it will have earned enough to enable it to replace the original plant at its own expense, and will have accumulating funds which will replace the later investments when they require replacement."

An analysis of his statement is in order:

The example given in Table 5 is that of a new plant costing $100 000, having a 10-year life, with the earnings for depreciation invested in additional plant. The total amount of capital invested by the owner from the start to the end of the tenth year is $100 000, but, on account of earnings for depreciation under the Equal-Annual-Payment Method invested in additions to the property, the total value new of the plant increases year by year from $100 000 at the beginning to $217 609 at the end of the ninth year.

Mr. Swain contends that the company is entitled to earn a fair return on the items in the last column—that is, on the value new of all the property, including the part paid for by earnings for depreciation—plus a depreciation allowance.

If, for the purpose of illustration, 7% were to be assumed as a proper annual return for the use of capital, a return of $7 000 each year for such use would meet all the requirements of equity. If, instead of taking 7% of the original investment, the property paid for by contributions from the rate-payers is also included, and all is valued as if new, the 7% per annum would amount to the following sums:

Year.	7% return.
1st	$7 000
2d	7 507
3d	8 085
4th	8 747
5th	9 502
6th	10 365
7th	11 350
8th	12 477
9th	13 763
10th	15 233
11th	16 911
12th	18 339

This tabulation shows that 7% of the value new would give a return of $15 233 in the tenth year, which would be 15.2% on the actual investment by the owner of the property.

The illustration assumes that additions to the property would be built out of earnings to the extent that the earnings sufficed for this purpose, thus deferring to the same extent the necessity for new capital. As a result, it would be necessary to supply $100 000 of new capital at the end of the tenth year, when, by the assumption, the original property would be replaced, and $7 238 of new capital at the end of the next year, to replace the property built out of earnings at the end of the first year. Mr. Stearns.

Strong objection has been raised by Mr. Swain and others to the suggestion that new capital may be used for renewals.

It would obviously be wrong to use new capital for this purpose if the additions purchased with the rate-payers' contributions for depreciation had been capitalized, as the capital account would be unduly swelled by so doing; but, where the increase of capital has been deferred by paying for additions out of earnings for depreciation, and, as a consequence, new capital is necessary to pay for renewals, the method is not inequitable.

The depreciation allowances invested in additions may represent a depreciation reserve as well as when invested in a fund, and, as a matter of accounting, it is feasible to draw from the depreciation reserve, however it may be invested, for making renewals, but, as a matter of fact and not of accounting, the money thus invested in additions is not available for renewals, but new money must be provided.

Reverting to the table under consideration, it will be seen that if the depreciation allowances invested in additions to the property are considered to be a depreciation reserve, they are sufficient at all times to maintain the investment intact and to pay for the renewal of the property when it reaches the end of its life. Therefore, earnings based on value new, over and above 7% on the amount of the actual investment by the owner of the property, represent a surplus which is not needed for the renewal of the property.

In the first 10 years alone, the accumulated surplus, using 5% compound interest, would amount to $38 308, or about three-eighths of the original investment, and this would be the measure of the confiscation of the property of the rate-payers by using value new in connection with the Equal-Annual-Payment Method of depreciation.

At several places in Mr. Swain's discussion he admits that the combined return for the use of capital and for the depreciation allowance, by the method recommended by the Committee, is fair when the rate of interest used in computing the depreciation allowance is the same as the rate of return for the use of capital. It would be difficult for him to do otherwise than to admit this, because the rates determined by the Committee's Equal-Annual-Payment Method, using the depreciated value of the property as the basis, are identical with

Mr. Stearns. the rates under corresponding conditions by the Sinking-Fund Method, using value new as the basis.

It is gratifying to have even this admission, because it shows that the use of the depreciated value of property is equitable under some conditions by the Committee's method. This being the case, how can it also be equitable to claim the much higher rates resulting from the use of value new?

Mr. Swain makes the claim that where the rate of interest used in computing the depreciation allowances is less than the rate of return on capital, the property of the stockholder would be confiscated. This claim is not a valid one.

The Committee's plan, as illustrated on page 34 of its report, provides depreciation allowances for amortizing items of property equal to 100% of their value, and a return at the assumed rate on all the investment in the item of property which has not been repaid by the rate-payers. The method, therefore, has incorporated in it all that is necessary to meet the fundamental equitable principles laid down by the Committee.

It is true that Column 7 of the table on page 34 indicates decreasing rates as the property grows older, but it does not follow that decreasing rates are inequitable to the owner of the property. If they were, the Straight-Line Method of depreciation would be especially inequitable, as will be seen by the rapid decrease in rates indicated by Column 7 of the table on page 40.

The fundamental difficulty with Mr. Swain's discussion of this subject seems to be his failure to recognize the difference between sinking-fund payments, which must be invested for the benefit of the sinking fund in order to complete it when an item of property reaches the end of its life, and depreciation allowances, like those provided by the Equal-Annual-Payment and Straight-Line Methods, which are a repayment of a part of the investment in the item of property and can be reinvested where they also will earn income for the benefit of the stockholder. When thus reinvested, whether in a fund or in the property, they keep the investment of the stockholder at all times intact and earning returns. No property of the stockholder is confiscated because of the transfer of a part of his investment from one item of property to another, or temporarily to a fund. The continuous ignoring of the returns from such reinvestment of the money returned as depreciation allowances appears to account for much of his adverse criticism.

Toward the latter part of his discussion, Mr. Swain offers an illustration to "make clear the fact that to take the cost-of-reproduction method, using the value-new-less-depreciation, may, in some cases, mean confiscation". He takes as an illustration a public utility property which is assumed not to need expenditures for renewals for

the first 10 years, and assumes that the amount earned above operating expenses and taxes varies "from 5% in the first year to 12% in the tenth and subsequent years". It is also assumed "that the depreciation is calculated on the Straight-Line Method, taking a life of 25 years, or 4% per annum". The further assumption is made that the original records of this property have been destroyed and that an appraisal is to be made at the end of the tenth year. Mr. Stearns.

He adds:

"The cost-of-reproduction method is used and shows a cost-of-reproduction-new equal to $1 000 000; but the commission says we must take the depreciated value and the depreciation is about 40%, so that as a basis for earnings, the value of the property is only $600 000. To this we may add the 12% in the renewal fund, or $120 000, making a total of $720 000."

He concludes:

"The value made on the basis of cost-of-reproduction-new-less-depreciation would confiscate the property of the stockholders, which was selling in the markets at $1 200 000 and justified that value, and now becomes worth only about $800 000, or perhaps less. If this is not confiscation, then the speaker does not know what confiscation is."

His Table 24, extended so as to cover the whole 25 years, is reproduced as Table 30.

TABLE 30.

Year of operation.	Percentage on investment earned for distribution to investors and for replacements and renewals.	Percentage paid to investors.	Percentage remaining for replacements and renewals.	Actual depreciation in percentage.
1st	5.00	5.00	0.00	4
2d	5.75	5.75	0.00	4
3d	6.50	6.50	0.00	4
4th	7.25	7.25	0.00	4
5th	8.00	8.00	0.00	4
6th	8.80	8.00	0.80	4
7th	9.60	8.00	1.60	4
8th	10.40	8.00	2.40	4
9th	11.20	8.00	3.20	4
10th	12.00	8.00	4.00	4
11th	12.00	8.00	4.00	4
12th	12.C0	8.00	4.00	4
13th	12.00	8.00	4.00	4
14th	12.00	8.00	4.00	4
15th	12.00	8.00	4.00	4
16th	12.00	8.00	4.00	4
17th	12.00	8.00	4.00	4
18th	12.00	8.00	4.00	4
19th	12.00	8.00	4.00	4
20th	12.00	8.00	4.00	4
21st	12.00	8.00	4.00	4
22d	12.00	8.00	4.00	4
23d	12.00	8.00	4.00	4
24th	12.00	8.00	4.00	4
25th	12.00	8.00	4.00	4

Mr. Stearns. It is "confusing" and "misleading" to have presented the matter in this way, because there is a complete omission of the item of development expense. Mr. Swain evidently believes that this expense should be added in making a valuation, because he gives a general endorsement of the conclusions of the Committee in all matters except those relating to depreciation, and these conclusions included a provision for development expense, and because he has stated in his discussion that "if there should be a lack of a fair return in early years, any such deficiency would be included under cost of developing the business, and in subsequent years a return would be allowed on this also".

According to Mr. Swain's assumptions, 12% is a fair return for the use and amortization of capital, and on this basis the deficiency of earnings in the first year would be 7%, in the second year, 6.25%, and so on, until the ninth year, when the deficiency would be 0.80%, with no deficiency thereafter. The sum of these deficiencies amounts to 35.5% of the original value of the property, and, with accumulations at 5% per annum compounded annually, would amount at the end of the tenth year to 48.6 per cent.

Assuming, as he has done, that the original cost of the property was $1 000 000, and that the reproduction-cost-new would also be $1 000 000, the reproduction-cost-new with the addition of the development expense, therefore, would be $1 486 000.

Of this reproduction cost, $111 000 would be on account of the deficiency in the amount paid to investors, and $375 000 on account of the deficiency in the sum for amortizing the property.

If the property continued to earn 8% on the original cost for distribution to stockholders for the remaining 16 years of its life, and 4% of the original cost as a depreciation allowance, the depreciation fund, including the sum to which the company would be entitled as a development expense under the foregoing assumptions, with interest at 5% compounded annually, would amount at the end of the twenty-fifth year to $1 909 000 to amortize property costing $1 000 000. This is another illustration of the confiscatory character of the Straight-Line Method of depreciation when used in connection with value new—confiscatory of the property of the public.

It is not surprising, in view of such a result, that Mr. Swain states that the Sinking-Fund Method is more favorable to the public. The illustration, however, does not warrant his additional statement that it is equally favorable to the corporation.

If, instead of considering the use of value new, the depreciated value of the property is taken, and, as suggested by Mr. Swain, an appraisal is to be made at the end of the tenth year, the cost of reproduction new is $1 000 000. From this would be deducted the total amount of depreciation by the Straight-Line Method, equal to 40%, giving, as the depreciated value of the property, $600 000. To this

would be added the 12% in the depreciation fund plus interest, amounting in all to $128 000, and the development expense computed on the basis set forth in the table, amounting at the end of the tenth year to $486 000, making a total of $1 214 000 for a property which originally cost $1 000 000. Mr. Stearns.

This looks very much like confiscation, but it is not the confiscation of the property of the stockholders, as claimed by Mr. Swain.

The enormous development expense in this case is due to two assumptions: First, that such a property is entitled to earn annually 8% of its original cost; second, to the use of the Straight-Line Method of depreciation in connection with value new.

A property of this kind is usually financed for a sum equal to at least one-half of its cost by the issue of bonds, which, in a case like that cited, could be sold on a 6% basis, and, with 8% return on the full value of the property, the stock would pay 10% to the stockholders.

For a 25-year property, if value new is to be used, the Sinking-Fund tables, using a 5% basis, give the proper amount to be earned annually on account of depreciation. This proper amount is 2.1 per cent. The annual depreciation charge of 4%, assumed by Mr. Swain, therefore, represents an excess of 1.9% annually on the full value of the property, either for distribution to the stockholders or for the formation of a surplus. If distributed to the stockholders, where the stock represents one-half of the value of the property, this would mean an addition of 3.8% to the 10%, making 13.8% available for dividends under the proposed assumptions.

If the excessive sums earned for depreciation by using the Straight-Line Method in connection with value new were not distributed to the stockholders, but were retained in a fund, they would amount at the end of the twenty-fifth year to $909 000, corresponding to the surplus already stated.

Mr. Swain's illustration will next be presented in a modified form, so as to make it more rational, and the Equal-Annual-Payment Method of the Committee will be used for computing depreciation.

It does not seem to be in accordance with public policy that the percentage paid to investors should be so large that a property costing $1 000 000 would be "bought and sold between investors on the basis of $1 200 000". Money could be obtained without doubt under such circumstances as those represented by one-half bonds earning 6% and one-half stock earning 8%, making the annual income on the whole value of the property 7%, and this amount will be used in place of the 8% in the previous illustration.

Table 31 covers 10 years, the same as Mr. Swain's Table 24. The percentage earned for the 10 years is copied from his table. The percentage paid to investors is the same in the first 3 years, and then 7%, instead of the larger sum. The percentage remaining for replace-

Mr. Stearns. ments and renewals amounts in the 10 years to 18.25 in Table 31, as against 12% in Table 24. The total amount of depreciation, using the Equal-Annual-Payment Method on a 5% basis, is 26.37%, in place of the 40% in Table 24. Table 31 shows that there would be a deficiency of earnings in the first 7 years and a surplus in the next 3 years. The development expense, assuming it to be the deficiency of earnings, including 5% interest, would amount to $187 000 at the end of the seventh year.

TABLE 31.

Year of operation.	Percentage on investment earned for distribution to investors and for replacements and renewals.	Percentage paid to investors.	Percentage remaining for replacements and renewals.	Actual depreciation in percentage.
1st	5.00	5.00	0.00	2.10
2d	5.75	5.75	0.00	2.20
3d	6.50	6.50	0.00	2.31
4th	7.25	7.00	0.25	2.43
5th	8.00	7.00	1.00	2.55
6th	8.80	7.00	1.80	2.67
7th	9.60	7.00	2.60	2.81
8th	10.40	7.00	3.40	2.95
9th	11.20	7.00	4.20	3.10
10th	12.00	7.00	5.00	3.25
Total for 10-year period.	84.50	66.25	18.25	26.37

If a valuation were to be made, as assumed by Mr. Swain, at the end of the tenth year, the value new would be, as he has stated, $1 000 000. The total depreciation to the end of the tenth year would be $264 000, leaving as the depreciated value of the property $736 000. To this should be added the 18.25% in the depreciation reserve plus interest, amounting in all to $200 000, and the development expense, assuming it to be the same as at the end of the seventh year, $187 000, making a total of $1 123 000 for a property which originally cost $1 000 000.

The adoption in this case of a more reasonable rate of return and a more reasonable depreciation allowance results in a more reasonable development expense and a more reasonable reproduction cost of the property. The reproduction cost, however, is still $123 000 more than the original cost, although so large an excess would not be warranted by the deficiency of dividends during the early years of the life of the property. There is still a confiscation of the property of the rate-payers, and this is due mainly to the assumption that during the first 10 years the earnings are based on value new and not on the depreciated value of the property, as they should be when this method of depreciation is used.

If an illustration had been presented in which the earnings were based on the depreciated value of the property, instead of on value new,

and if 7% compound interest were used in computing the depreciation allowance, the development expense and the accumulation of the depreciation reserve, the reproduction value of the property, including the depreciation reserve and the development expense at the end of the tenth year, would have been only enough more than $1 000 000 to compensate for the net deficiency of dividends during the 10-year period. Under such circumstances, there would be no confiscation of the property of either the corporation or the rate-payer. Mr. Stearns.

The Committee holds the view that the appreciation of land, and of some other items of property which appreciate in value, is closely related to the depreciation in the value of the structural property, and that, if the public makes good to the corporation the loss of value of the portions of the property which in the course of events are practically sure to depreciate, the corporation should also make good to the public the increase in value of the portions of the property which are practically sure to appreciate.

Mr. Swain takes the ground that an appreciation in land is an appreciation in price. He mentions two kinds of depreciation, one due to wear and tear and approaching obsolescence and the other which may result from the smaller cost of performing work, and states that the appreciation in land should be compared with the depreciation due to the smaller cost of performing work and not to the depreciation caused by wear and tear and approaching obsolescence.

Under existing laws it seems to be necessary to value property at substantially present prices, and the increase or decrease in value, due to changes in the cost of performing work, which, in recent years, has been almost wholly increase, is a risk which the owner of a property under existing conditions is obliged to assume.

In order to obtain equitable results, the cause of a continuing depreciation or appreciation in the value of property is not important. It is the fact that structural property is bound to depreciate which makes it equitable for the public to make good to the corporation annually the amount of such depreciation, and, similarly, the fact that land under most circumstances is bound to appreciate in value which makes it equitable for the corporation to make good to the public the amount of such appreciation.

Owing to a rise in prices, items of property which in time are bound to go out of use owing to wear and tear or obsolescence may be worth more after they have attained a certain age than when new, and, similarly, owing to change in prices, there may be years in which land will not appreciate, but, in the long run, land in a growing community does appreciate in value.

Mr. Swain takes the ground that in appraisals the present value of the property must be taken, and that, if the reproduction cost at present prices means an increase in the value of the property, the corporation is entitled to the increment.

Mr. Stearns.

In this argument, however, he does not take the present value of the perishable property, but the value of such property as it would be if it were new, although it is obvious that an old locomotive, car, or tie has much less real value than a new one. If he were to take the real value of such perishable property, instead of its value new, the same line of reasoning, by which he attempts to show that the corporation should not be required to make good to the public the amount of the appreciation in land, might be used to show that the public should not be required to make good to the corporation the depreciation in the value of the perishable property.

The most common argument against the Committee's position that equity requires the owner of a public service property to account to the public for the appreciation of land is expressed by C. P. Howard,* M. Am. Soc. C. E. He states that "All other owners of property enjoy this increase as a clear bonus; the unearned increment". Is this a correct statement?

In the case of a public utility, the owner should be permitted to charge rates to reimburse the sums paid for taxes on the land, and, under the views expressed by the Committee, he is entitled to have a fair return on the fair value of the land as well as on the fair value of any other property.

The private land owner does not have his taxes paid for him and is not entitled to receive from the public a fair annual return on the value of his property. In most cases where the land is vacant he does not receive more than a meager return, if any, and has to depend mainly on the appreciation in the value of the land in order to obtain finally a fair return.

If it is correct to infer that because private owners enjoy an increase in the value of land, the owner of a public utility should likewise enjoy such increase, is it not equally correct to infer that if the private owner of a building on the land has to stand the loss due to the decrease in value of the building as it grows older, the owner of the public utility should likewise stand the loss due to the decrease in value of a building owned by him, and not claim that the public should pay him a depreciation allowance to compensate him for such loss?

To revert once more to Mr. Swain's discussion, he claims that in the case where the cost of constructing a property is known, the cost new is an admittedly correct method, and that as the cost-of-reproduction-new is a substitute for cost new, when the latter cannot be ascertained, it also should be used instead of the depreciated value.

If cost new were an admittedly correct method, then the deduction made by Mr. Swain would be logical, but, in the speaker's opinion, it is not a correct method when a method of providing for depreciation is used which returns to the owner of the property the full value of each item during its life.

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* *Proceedings*, Am. Soc. C. E., for February, 1914, p. 387.

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PAPERS AND DISCUSSIONS

STEEL STRESSES IN FLAT SLABS.
Discussion.*

BY MESSRS. L. J. MENSCH, W. K. HATT, AND H. T. EDDY.†

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—This paper appears to the writer to be a defense of, and a propaganda for, the so-called mushroom type of flat floor construction. Mr. Mensch.

The author seems to be quite satisfied that mushroom floors with a notoriously deficient quantity of steel reinforcement are safe and proper constructions. This is a debatable question, with reference to the factor of safety required. In some constructions there is a factor of safety of a little more than 1, as in foundations, retaining walls, dams, bridge abutments, etc., in fact, where the carrying capacity and cohesion of the ground is to be considered. In other cases a factor of safety of hardly 2 is often, knowingly or unknowingly, adopted, as in columns, whether of steel, brick, or concrete, especially in outside columns and those which are eccentrically loaded, also in roof trusses, common brick walls in buildings, etc.

The numerous cases where such low factors of safety are found may blind an engineer and make him believe that he is as well justified in adopting a similar factor of safety in flat slab construction. On the other hand, engineers are accustomed to allow a factor of safety of 3 in steel-girder constructions, and the general public demands a factor of safety of at least 4 in reinforced concrete construction, on account of its novelty and its greater uncertainty in erection.

The writer claims that a large percentage of the examples cited by the author have a factor of safety of a trifle more or less than 2, and that his formulas simply accommodate the extensometer tests of these slabs. Extensometer tests do not give any indication of the real strength of a slab, which is reinforced by light bars and to such low amounts as ¼ per cent. The extensometer gives the elongation in a

* Continued from March, 1914, *Proceedings*.

† Author's closure.

Mr. Mensch.

length of generally 8 in. of steel bars embedded in concrete. Where the surrounding concrete is not cracked in the 8 in., the extension obtained by the extensometer, in inches per inch of length, multiplied by the modulus of elasticity of the steel, generally taken at 30 000 000, will give the steel fiber stress, but no correct idea of the interior resisting moment, because the tensile strength of the concrete is disregarded. Assume, however, that there is a crack in the concrete between the contact points of the extensometer, then the stress is transferred by shear and by the bond of concrete and steel to the steel bar in the neighborhood of the crack, and the bar is stressed considerably higher at the crack than at points farther from it.

For an illustration, assume that the reinforcement consists of $\frac{3}{8}$-in. round bars, 6 in. from center to center, that the crack in the concrete is 1 in. deep, and that the tensile strength of the concrete is 200 lb. per sq. in. The surface of a $\frac{3}{8}$-in. bar per lin. in. is 1.18 sq. in., and its cross-section is 0.11 sq. in. The original strength of a concrete section, 6 in. wide and 1 in. deep, is $6 \times 200 = 1\,200$ lb., and, assuming a bond stress of 400 lb. per sq. in., or 472 lb. per lin. in. of $\frac{3}{8}$-in. bar, it will require a length of 3 in. on each side of the crack to transmit the original strength of the cracked portion of the concrete into the bar. This additional stress in the steel fibers amounts to $1\,200 \div 0.11 = 10\,900$ lb. per sq. in., and, for determining the elongation of the bar we have to assume that it acts only on an average of 3 in., though the extensometer measures on 8 in.; hence the stresses obtained by extensometer readings are in this particular case only three-eighths of the actual stresses. In other words, when one tries to judge, from extensometer readings, the distribution of the exterior bending moments, or their equivalent, the interior resisting moments, one is likely to under-estimate the value of the exterior bending moments 50% and more, due to the neglect of the tensile stresses of the concrete.

The fact that extensometer readings do not permit any correct conclusions as to the interior resisting moment, or the factor of safety of the slab, is indisputably proved, by tests made by Richard L. Humphrey, M. Am. Soc. C. E.* In that investigation there were 336 beams, 13 ft. long, 8 by 11 in. in cross-section, and they were tested on a 12-ft. span by two equal loads at the one-third points. The reinforcement throughout consisted of $\frac{1}{2}$-in. bars, having a yield point of approximately 40 000 lb. The number of bars varied from two to eight, corresponding to a percentage of $\frac{1}{2}$ to 2. The mixture of the concrete was 1 : 2 : 4 by volume, and the tests were made when the beams were 4, 13, 26, and 52 weeks old. In every beam tested there were careful readings of the deformations, both of the top fibers and of the steel fibers, for stages of loadings of from 500 to 1 000 lb. up to the ultimate load.

* Technologic Paper No. 2, U. S. Bureau of Standards.

TABLE 14.—LOG SHEETS OF GRAVEL CONCRETE BEAMS, 13 WEEKS OLD. Mr. Mensch.

Applied load, in pounds.	$\frac{M}{bd^2}$, in pounds per square inch.	Deformation, in Millionths of an Inch per Inch. Upper fiber.	Deformation, in Millionths of an Inch per Inch. Steel fiber.	K, percentage.	Deflection, in inches.
Beam 337; ½% Reinforcement.					
0	24.85	17	21	44.6	0.010
1 000	54.85	48	44	52.0	0.020
2 000	84.85	66	63	51.3	0.030
3 000	114.85	112	115	49.4	0.050
3 500	129.85	138	156	46.9	0.060
4 000	144.85	172	236	42.2	0.080
4 500	159.85	237	472	33.5	0.130
5 000	174.85	282	639	30.6	0.170
5 500	189.85	319	815	28.1	0.215
6 000	204.85	359	993	26.5	0.270
6 500	219.85	398	1 162	25.5	0.310
6 740	227.05	....	1 241	......	0.340
5 600	192.85	491	1 154	29.9	0.340
Beam 338; ½% Reinforcement.					
0	24.64	21	19	52.5	0.010
1 000	54.64	51	43	54.1	0.025
2 000	84.64	75	63	54.3	0.035
3 000	114.64	115	109	51.4	0.055
3 500	129.64	142	140	50.4	0.060
4 000	144.64	182	217	45.6	0.085
4 500	159.64	235	373	38.6	0.120
5 000	174.64	286	566	33.5	0.170
5 500	189.64	337	797	29.7	0.225
......	....	....	....	......	....
5 000	174.64	411	1 684	19.6	0.330
Beam 348; ¾% Reinforcement.					
0	25.26	23	15	60.4	0.010
1 000	55.26	53	38	58.0	0.025
2 000	85.26	77	56	57.8	0.035
3 000	115.26	113	85	57.0	0.050
3 500	130.26	126	103	55.1	0.055
4 000	145.26	146	127	53.4	0.060
4 500	160.26	167	168	49.8	0.070
5 000	175.26	209	253	45.3	0.100
5 500	190.26	257	362	41.5	0.120
6 000	205.26	293	472	38.3	0.150
7 000	235.26	357	656	35.2	0.210
8 000	265.26	421	865	32.8	0.270
9 000	295.26	481	1 072	31.0	0.330
9 370	306.36	....	1 154	......	0.355
9 620	313.86	....	1 338	......	0.390
8 000	265.26	534	1 726	23.7	0.395

Applied load, in pounds.	$\frac{M}{bd^2}$, in pounds per square inch.	Deformation, in Millionths of an Inch per Inch. Upper fiber.	Deformation, in Millionths of an Inch per Inch. Steel fiber.	K, percentage.	Deflection, in inches.
Beam 349; ¾% Reinforcement.					
0	25.47	13	15	47.2	0.010
1 000	55.47	42	39	51.9	0.020
2 000	85.47	64	60	51.7	0.040
3 000	115.47	93	84	52.5	0.050
4 000	145.47	117	111	51.4	0.065
4 500	160.47	145	149	49.3	0.080
5 000	175.47	171	207	45.2	0.100
5 500	190.47	217	321	40.3	0.120
6 000	205.47	257	456	36.0	0.150
7 000	235.47	321	706	31.3	0.220
8 000	265.47	380	913	29.4	0.280
9 000	295.47	433	1 087	28.5	0.330
9 500	310.47	464	1 197	27.9	0.360
10 000	325.47	489	1 294	27.4	0.390
10 500	340.47	516	1 378	27.2	0.415
11 000	355.47	551	1 467	27.3	0.440
11 050	356.97	....	1 516	......	0.465
9 300	304.47	548	1 981	21.7	0.475
Beam 350; ¾% Reinforcement.					
0	25.06	19	21	47.5	0.005
1 000	55.06	43	46	48.6	0.020
2 000	85.06	70	68	49.6	0.030
3 000	115.06	102	96	51.5	0.040
4 000	145.06	133	130	50.6	0.050
4 500	160.06	160	169	48.7	0.060
5 000	175.06	198	248	44.4	0.080
5 500	190.06	244	390	38.5	0.115
6 000	205.06	289	533	35.2	0.150
7 000	235.06	360	754	32.3	0.210
8 000	265.06	418	945	30.7	0.265
8 800	289.06	....	1 250	......	0.345
7 500	250.06	498	1 597	23.8	0.355
Beam 360; 1% Reinforcement.					
0	25.26	23	17	57.5	0.010
1 000	55.26	56	43	56.5	0.015
2 000	85.26	84	65	56.5	0.025
3 000	115.26	107	87	55.1	0.035
4 000	145.26	135	111	54.9	0.045
4 500	160.26	158	130	54.9	0.050
5 000	175.26	181	162	52.8	0.060
5 500	190.26	206	222	48.2	0.080
6 000	205.26	246	330	42.7	0.100
7 000	235.26	309	508	37.8	0.150
8 000	265.26	358	643	35.8	0.190
9 000	295.26	409	786	34.2	0.230
10 000	325.26	455	938	32.6	0.275
11 000	355.26	508	1 068	32.2	0.320
12 000	385.26	556	1 200	31.7	0.360
12 850	410.76	....	1 362	......	0.400
10 700	346.26	626	1 921	24.6	0.430

Mr. Mensch.

In Table 14, taken from Technologic Paper No. 2, it will be noticed that Beam No. 337 failed at a total load of 6 740 lb. At a load of 3 500 lb., which is more than one-half of the ultimate load, the extensometer reading of the steel fiber was $\frac{156}{1\,000\,000}$ in. per in.; or, multiplying this extension by 30 000 000, we obtain a stress of only 4 680 lb. It is well to keep in mind that the extensometer reading gives such a low stress when the factor of safety is less than 2. Similarily, we find that at a load of 4 500 lb., which is two-thirds of the ultimate load, the stress in the steel fibers is 14 160 lb. At 5 000 lb., which is 75% of the ultimate load, the steel fiber stress is only 19 000 lb. per sq. in. Similar conditions may be found in every beam tested, even in those with 2% of reinforcement. One may reasonably expect that the discrepancies will be still greater in flat slab construction, where the percentage of reinforcement is considerably lower than in Beam No. 337, being mostly only $\frac{1}{4}$ to $\frac{1}{5}$ of 1 per cent. One can even imagine the case where, with an insufficient quantity of reinforcement, the slab will break at practically the same load as a non-reinforced slab, where, just before breaking the extensometer would not show a larger extension that 0.0001 in. per in., corresponding to a stress at failure of only 3 000 lb. per sq. in.

In studying the tables it is found that at certain stages a very small increase of the load causes a great change in the extensometer readings, which explains the great discrepancies found by Mr. Eddy in the stresses of adjoining bars or of symmetrical points. The unreliability of extensometer readings for the determination of the interior resisting moment of reinforced concrete beams was discussed by the writer before the Western Society of Engineers in the spring of 1904.*

There is yet another simple indication that the flat slab constructions cited by Mr. Eddy have not the conventional factor of safety, and that is the appearance of cracks at the most dangerous sections, where one would expect to find them at loadings of from one-third to one-half of the ultimate load. From tests of beams reinforced by $\frac{1}{4}$%, it is known that cracks appear at a load which is about three-fourths of the ultimate; and, in slabs reinforced by $\frac{1}{2}$%, at about one-half of the ultimate load.

Mr. Eddy writes:

"Indeed, slab action in general may be described partly as the attempted mechanical superposition of one set of parallel depressions and elevations on another set of similar corrugations at right angle to them. Such sets mutually support each other and give rise to slab action. * * *"

This sounds mysterious, and will not enlighten any one; it only shows that the author does not care to express in a scientific way the

* The *Journal* of the Western Society of Engineers, for 1904, contains a number of tables and diagrams.

real slab action, as has been done, for example, in a masterful way, by the Italian engineer, Danusso,* and by others. Mr. Mensch.

There is no mystery about slab action. It is simply a combination of continuous beams acting in at least two directions, which beams are connected with each other and with the columns, and the deflection in any point is common to at least two beams acting at right angles to each other.

On account of the large sizes of the columns and column heads, and the depression of the floors at the columns, the beams directly connecting the columns, and corresponding to the sides of the squares, have considerably greater stiffness than those parallel to them and nearer the center of the slab, and these beams have a comparatively still greater stiffness than any diagonal beams, with the result that the mechanical action of the whole slab must be nearly identical with the action of a flat slab of a smaller span than the distance from center to center of columns, supported by four wide girders of a depth somewhat greater than the thickness of the slab.

If we omit to take into account the presence of the large columns and column heads, we obtain the case considered by Winkler and Grashof, who assumed the supports of the slab to consist of ideal points, and who found that the bending moment in the center of the span is $\frac{WL}{48}$ and over the columns and side of the square is $\frac{WL}{24}$, when W is the total load on the panel, L is the distance from center to center of columns, and Poisson's ratio is 0.1. The bending moments change only very slightly if other values of Poisson's ratio are assumed. For steel, it is generally found at 0.3, and the moments become $\frac{WL}{26.4}$ and $\frac{WL}{52.8}$, respectively. Returning again to the flat slab as a combination of four wide beams supporting a smaller flat slab, it is only within reason to assume that the wide girders have to carry the same loads as ordinary beams in the case of a flat slab supported by narrow and deep beams.

It is here that one finds a riotous license of figuring by most of the advocates of flat slab constructions. According to most building ordinances and accepted good practice, the bending moments in beams supporting flat slabs must be taken both over the supports and in the center of the beams at not less than $\frac{W}{2} \times \frac{L}{12} \times \frac{3}{2}$ (the latter factor on account of the triangular application of the load over the beam) $= \frac{WL}{16}$, although theory shows that the bending moment in the center of the span is only $\frac{WL}{32}$.

* "Il Cemento," 1912.

Mr. Mensch. Now, the advocates of the mushroom system are not satisfied to figure the bending moment in the center of the side beams as the theoretical moment, which is only one-half of the moment any engineer is allowed to figure when he adopts common girder construction; they go still further, and claim the right (according to Mr. Eddy's Equation 34) to figure this moment as $\frac{W L}{175}$, which is one-tenth of the moment required by the building ordinances. There we hear again that they must be right or their constructions would fail at the first application of the design load, and that they are right because the extensometer readings bear them out. The writer has shown how erroneous are the results obtained by extensometer readings; it remains yet to prove that the real bending moments in the center of the side beams are less than are required by the building ordinances, otherwise their structures actually would not stand up.

In flat slabs supported by continuous girders of slightly greater stiffness than the slab, the Italian engineer, Danusso, has shown that the load of the slab is distributed nearly uniformly over the girders, instead of with the triangular distribution found for very stiff beams; hence the bending moments for a beam supporting two adjoining panels are theoretically $\frac{W L}{24}$ and $\frac{W L}{48}$, at the supports and the center, respectively. Engineers need not be surprised that the bending moments for the stiffer side beams are just as large as those given by Grashof for the whole width of the slab. This case happens very often in ordinary girder and joist construction. Assume, for example, a flat slab supported by parallel walls, 20 ft. from center to center. If this distance is spanned by a simple slab, the bending moment is $\frac{W L}{8}$. As a rule, it is cheaper to provide girders of 20 ft. span and, say, 10 ft. from center to center, which girders must be figured for a moment of $\frac{W L}{8}$, and the slabs between the girders must be figured for another bending moment. Hence the sum of all bending moments is larger than the original moment $\frac{W L}{8}$, yet the cost of the construction is generally less.

In continuous beams which are of larger moment of inertia near the supports than in the center, such as is the case with the large columns and column heads used, it is known, also, that the moments at the supports become larger and those in the center of the span become smaller; taking also into consideration the fact that the span of the side beams may be safely diminished by one-half of the diameter of the columns, the moments of $\frac{W L}{24}$ and $\frac{W L}{48}$ become in most cases

cited by Mr. Eddy, where no depression in the floor is used, $\frac{W\,L}{24.4}$ and $\frac{W\,L}{62}$; and, where a depression is used, they become $\frac{W\,L}{22.2}$ and $\frac{W\,L}{92}$. Hence, in the most favorable case, giving the mushroom system all the advantages of continuous action, such as is not adopted by any responsible designer in any other class of work, neglecting at the same time all possible settlements, Mr. Eddy advises the use of a bending moment of one-half of what it can possibly be, which readily explains the low factor of safety. Mr. Mensch.

The advocates of the mushroom system may claim that the diagonal beams help to support a great portion of the load which the side beams are assumed to carry. Mr. Eddy, however, admits that the stresses in the center of the diagonal beams are generally found to be one-half as great as those in the side beams; this, together with the fact that the span of the diagonal beams is 1.41 times that of the side beams, permits the conclusion that a diagonal beam supports only one-quarter of the load of a side beam, and, in fact, simply transmits the loads from the interior of the panel to the side beams.

Mr. Eddy advises calculating the diagonal beams in the center for a moment of about $\frac{W\,L}{250}$, which does not seem to be sufficient, even for the simple transmission of the loads to the side beams. Granting that the width of the side beams may be assumed to be four-tenths of the span from center to center of columns, the width of the interior slab is six-tenths of L and the bending moment per linear foot of the interior slab, according to most building ordinances, must be figured as $\frac{w\left(\frac{6}{10}L\right)^2}{24} = \frac{w\,L^2}{66.67}$, while $\frac{W\,L}{250}$ per diagonal beam corresponds to $\frac{w\,L^2}{150}$ per lin. ft. of the interior slab.

It is remarkable that Mr. Eddy overlooked the fact that the distance between the points of inflection on the sides of the squares of several of the tests cited by him were found to be from $0.5L$ to $0.55L$. According to his own statements, the bending moment must be larger in the center of the sides than $\frac{w.(0.5\,L)^2}{8}$ and $\frac{w.(0.55\,L)^2}{8}$, or $\frac{w.L^2}{32}$ and $\frac{w.L^2}{27}$, respectively, while his moment of $\frac{W.L^2}{175}$ corresponds to $\frac{w.L^2}{70}$.

The advocates of many flat slab systems declare that the common beam theory does not apply to flat slabs. The writer has never seen a similar statement in the works of St. Venant, Winkler, Grashof,

Mr. Mensch. Föppl, or Müller-Breslau, who, in much more difficult problems than this, applied only the common beam theory, and in particular, the case of flat slabs was solved by Winkler and Grashof by that theory. There is yet one defense open to the advocates of the mushroom system, and that is the mystic influence of Poisson's ratio, but Mr. Eddy himself states that, on account of bending moments of opposite signs acting at right angles at the center of the side beams, the stresses there are larger than in a common beam; hence he can use it only for a defense of the low moments he assumes over the columns, and he claims that Poisson's ratio must be assumed as one-half. This, however, is a mechanical impossibility, clearly excluded by Grashof, as a simple change in shape without any change in the distance of the molecules is identical with a Poisson's ratio of one-half. Over the columns we have the most unfavorable combination of maximum bending moment and maximum shear acting together in the same section, and Grashof advised, even in the case of steel, reducing the stresses over the columns by 20 per cent. Besides, near the ultimate load, the concrete is cracked in every direction around the columns, and the steel bars will hardly be benefited by stresses acting in two directions, although the concrete in compression may be helped considerably. The percentage of reinforcement being low, an increase of the compressive stress of the concrete of 50% will benefit the internal resisting moment less than 5 per cent.

Mr. Eddy considers only the case of all panels loaded, which is a more favorable one for the center of all beams than that of a single panel, or two panels loaded, and extensometer tests clearly bear this out. In the discussion of a former paper on this subject,* the writer has shown that in the latter case the slab and the supporting columns and the columns above the slab form an arch construction in which the columns are very highly strained by the negative moments at the supports, and that the negative moments in the adjoining panels are very small when the columns have a larger moment of inertia than the slab. These conclusions were proved indisputably by the tests recently made by Mr. W. A. Slater, of the University of Illinois, on a new factory building of the Shredded Wheat Company, at Niagara Falls, N. Y. Mr. Slater presented the results of the test in a paper before the meeting of the American Concrete Institute in Chicago, in February, 1914. Although the floors were tested to only 1½ times the total dead and live load, numerous fine cracks appeared, and the extensometer indicated steel stresses of 15 000 lb. per sq. in. in the slab and additional compressive stresses in the columns, due to the loading of panels on one side of the columns, of 400 lb. per sq. in.

Mr. Eddy declares that the drops in the floors are unsightly, bulky,

* *Proceedings*, Am. Soc. C. E., for August, 1913.

and unnecessary, and prefers the mushroom type, where the depression of the floor is omitted and replaced by a magic steel ring in the neutral axis of the slab. That this advocates very poor construction, not countenanced by any reputable engineer, may be seen from the detail of the slab of the Northwestern Glass Company's Building, cited by Mr. Eddy. The column caps scale 36 in. in diameter, the dead weight of the floor is 120 lb. per sq. ft., the live load is 400 lb. per sq. ft., and the total load of a panel supported by one column is $16 \times 17 \times 520 =$ 141 440 lb. The floor slab is 8 in. thick, and the sectional area of the slab in shear around the cap equals $36 \times 3.14 \times 8 = 904$ sq. in., or a shear of $141\,440 \div 904 = 155$ lb. per sq. in. of the total section, or of about 220 lb. per sq. in. for a section of the depth of jd. Most building ordinances allow only a shear of 125 lb. per sq. in. on the section of a depth of jd, and prescribe the use of bent-up bars and stirrups at the section of great shear; on the other hand, the advocates of the mushroom system and their imitators take another license to step over ordinances and sober reasoning, allow a high shear, and no scientific reinforcement for shear whatsoever. Mr. Mensch.

When called to task for allowing such a high shearing stress, these experts defend it by saying that only "punching shear" is acting. The writer does not know that "punching shear" is a term common in engineering, but, if it means anything, it must mean pure shear, like that in a true punching operation, when practically no bending stresses are acting on the material which is being punched. We have shown before that around the columns the maximum moment and the maximum shear are acting together, and that the stresses must be reduced and not increased at that section.

A great many thoughtless and entirely erroneous statements in reference to flat slabs may be found in most articles published on this subject in current engineering literature, and an example is Table 15.

TABLE 15.

No.	Method.	Thickness of slab, in inches.	Steel per panel, in pounds.
1	Cantilever	8	2 189
2	Turneaure and Maurer	12	1 931
3	Grashof	8	784
4	Mensch	8	2 120
5	Turner	8	549
6	McMillan	8	1 084
7	Brayton	8½	1 900

This table is from a discussion by A. W. Buel, M. Am. Soc. C. E., in *Proceedings*, Am. Soc. C. E., for August, 1913.

From Table 15 it would appear that a flat slab, of 20 ft. span and a thickness of 8 in., contains per panel 784 lb. of reinforcing steel, if designed according to Grashof for a total dead and live load of 300

Mr. Mensch.

lb. per sq. ft. The writer has shown that Grashof's moments for the center and the sides of the square are $\frac{W.L}{48}$ and $\frac{W.L}{24}$, $W = 400 \times 300 = 120\,000$ lb., $L = 20$ ft., and $\frac{W.L}{48} = 50\,000$ ft-lb. and $\frac{W.L}{24} = 100\,000$ ft-lb. for the entire width of the panel, or per lin. ft., 2 500 ft-lb. and 5 000 ft-lb., respectively.

Adopting an allowable fiber stress for the reinforcing steel of 13 000 lb. (although it is too high for mild steel where a factor of safety of 4 is desired), and assuming the distance of center of reinforcing to the compression face as 7 in. (which is a trifle too small for the center of the slab and considerably too large for the sides of the square), we find that the sectional area of the reinforcement for a bending moment of 2 500 ft-lb. per lin. ft. in the center of the slab is 0.33 sq. in. and double the area is required at the sides of the square.

Where the negative reinforcement is obtained by bending all bars up at the quarter point, and continuing the rods to the quarter point in the adjacent panels, the weight of the steel reinforcement per linear foot of the panel in one direction is $0.33 \times 3.4 \times (20 + 2 \times 5) = 34$ lb., and for the entire panel in both directinos $34 \times 20 \times 2 = 1\,360$ lb. It is very poor practice to bend up all bars; this would not be permissible in any girder construction on account of reversal of moments which may occur by settlements and unfavorable loadings in other panels. Assuming that one-third of the bars are straight and extend only 1 ft. beyond the center line at each end, and that, therefore, extra rods are required on top over the sides of the square, we find that at least 340 lb. of additional reinforcement must be placed in each panel, or a total of 1 700 lb.

Hence, according to Grashof, it requires in continuous panels at least three times as much steel as used in the Turner system, provided that the statement in Table 15 in regard to the Turner system is correct. Inasmuch as we have to design for the case of only two panels loaded side by side, it requires a great deal more reinforcing than the 1 700 lb., not counting the additional reinforcing which is absolutely necessary in the columns to overcome the bending stresses produced by one-sided loadings.

The writer begs not to be misunderstood as being an opponent of flat slab constructions. He thinks he was the first to use flat slabs on a large scale, for he built two slabs, 100 ft. square, at the new plant of the Proctor and Gamble Company's soap works at Armourdale, Kans., in 1903, and has built many others since. He protests, however, when engineers and contractors (who, as a rule, have all to gain and nothing to lose), represent by versatile agents that flat floor constructions, of designs as advocated by Dr. Eddy, are as good as, or

better than, girder constructions, which actually have a factor of safety of 4 or more, if they are designed according to the Chicago Building ordinance, for example, and if they are constructed right. No wonder that they can show to unsuspecting architects and owners a great saving over all other designs, and, being able to mention a great number of examples of buildings which did not fall down (having a factor of safety of about 2), are believed to be by the owners and architects very wizards in the art of reinforced concrete construction. It is true, however, that flat slabs show a slight saving (even if they are designed for a factor of safety of 4) over girder construction of the same factor of safety. The writer has not the reputation of wasting any material in the structures which he designs and builds; being a contractor, it would simply diminish his profits, yet he can mention a number of examples which were actually loaded with about four times the total dead and live load without breaking down. Mr. Mensch.

In December, 1902, a girder in the Salvation Army Building, in Cleveland, of 24 ft. span and designed for a load of 100 lb. per sq. ft., was tested with 100 000 lb., which is about $3\frac{1}{2}$ times the total dead and live load. The girder showed some cracks, but no indication of a near failure, and the permanent deflection was $\frac{1}{4}$ in.

In the spring of 1905 the writer built a furniture storage house for the Brown Transfer and Storage Company, at St. Joseph, Mo. The structure was designed for a live load of 100 lb. per sq. ft. In the fall of 1905, when the writer started another building for the same company, he found that the building was rented to the Parker Grocery Company, which lost its own building by fire, and was loaded all over to the ceiling with canned goods weighing at least 600 lb. per sq. ft., so that both floors and columns withstood nearly four times the total dead and live loads.

In 1905 the writer designed and constructed a building for the Seng Company, Dayton Street, Chicago. The building was designed, according to the new Chicago ordinance for reinforced concrete, for a live load of 100 lb. per sq. ft. On account of some failures of concrete buildings all over the country at that time the owner refused payment on the building as the work progressed until such time as the writer could prove that the building had a factor of safety of 4. In the presence of a great number of engineers and architects, the writer tested one panel, only one month old, in December, 1905, with 600 lb. per sq. ft. The girders and slabs deflected and cracked considerably, but did not fail under the load, which remained there 24 hours. The permanent deflection was only $\frac{1}{8}$ in.

These examples should suffice to show that girder and slab constructions, correctly designed and built, have a factor of safety of at least 4, and there is no question that flat slab constructions can be made as safe as any others, but the writer claims that flat slabs designed according to Mr. Eddy's formula have only a factor of safety of 2, under the most favorable circumstances.

Mr. Hatt. W. K. HATT, M. AM. SOC. C. E. (by letter).—The recent extension of testing operations to completed structures has shown, what many have known, that reinforced concrete structures carry their working loads in service by the help of tensile stresses in the concrete. The stresses in the steel may not be more than one-half or one-third of those which would be calculated by the ordinary formulas, which neglect these tensile stresses.

For the purpose of standardizing design, and as a matter of safety, the conventions of the building codes properly omit these tensile stresses. They cannot be ignored, however, when the complete action of the structure is to be accounted for under working loads.

The variability of the quality of concrete not only makes it desirable to omit these tensile stresses in calculations for design, but it also renders the steel stresses, which are measured in tests of buildings, a very uncertain factor; that is to say, the measured stresses in the steel depend very largely on the quality of the concrete. Good concrete may relieve the steel of the greater part of its expected moment of resistance.

It follows, therefore, that, in judging the mechanical action of any type of structure, more attention should be paid to the measured compressive stresses in the concrete than to the measured stresses in the steel.

Mr. Eddy. H. T. EDDY, ESQ. (by letter).—Mr. Godfrey has taken the opportunity to repeat his objections and warnings against the use of flat slabs, and especially to insist that, whatever may be the true theory of an interior panel of a flat slab of indefinite extent and uniformly loaded, that theory cannot apply either to a single loaded panel or to an exterior panel, and, further, that the theory under discussion involves reliance on direct tension stresses in the concrete.

The full treatment of these questions involves a multitude of details, including bending moments in columns, cantilever action, direct and indirect stresses in concrete, bond shear, shock due to moving loads, the conditions under which beam action prevails over slab action, etc., etc. These questions and criticisms have been raised before by Mr. Godfrey, in connection with the discussion of Mr. Nichols' paper,* and have been answered in detail. Ultimately, dependence must be placed on the facts as they appear in test and practice, rather than on *ex cathedra* statements of what must occur as a consequence of views held on theoretical questions.

The crux of the entire matter seems to lie in the question of direct and indirect stresses in concrete, the latter being those called into play by bond shear and the former by bending.

* "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors," *Proceedings*, Am. Soc. C. E., for April, 1913.

(This paper has not yet been published in *Transactions*, owing to the failure of the author to close the discussion.)

The indirect tensile and compressive stresses in the concrete due to bond shear enable the rods of the belts which cross each other to co-act with each other under tension and cause the slab to exhibit effects similar to those measured in flat plates by Poisson's ratio. It is evident that as long as the bond is intact at the surfaces of the rods, the indirect stresses—which necessarily have the same numerical intensity as the bond shear—do not exceed the strength of the concrete, and the mechanism for the same kind of interaction exists in the slab as in the plate. This phenomenon, therefore, must be taken account of in one case as much as in the other. Consequently, the value of K for slabs is inextricably bound up with the question of bond shear and indirect stresses in the concrete in the tensile zone. No general discussion of the numerical value of K will be taken up at this time, other than to remark in passing that, in order that a homogeneous solid may remain of constant volume, it is necessary from geometrical considerations to have $K = 0.5$, a value which undergoes greater or less diminution in case of various actual solids, according to their volumetric elasticity. Mr. Eddy.

Likewise, in order that a material sheet or surface may remain of constant area, it is geometrically necessary that $K = 1$, where K refers to lateral distortion in the plane of the sheet or surface, as it does in a slab, and has nothing to do with changes of thickness of the sheet. For sheets of various materials, this value of K will be diminished according to its areal elasticity in resisting forces tending to stretch it in its plane.

All reinforced concrete construction is based on the effectiveness and reliability of bond shear. Without its action, no beam or slab could endure, but its action in a slab, not only effects all the results it produces in a beam, but must necessarily cause the slab to exhibit additional properties due to the interaction of multiple-way reinforcement consisting of wide belts of rods in contact with each other. In case the belts are made of numerous parallel rods with a spacing comparable to the thickness of the slab, the slab is sufficiently fine-grained to act very much as if it were of uniform texture but formed of some material not the same as either of its constituents.

It is entirely beside the mark for Mr. Godfrey to argue that, because this composite material has in it reinforcing rods, and Poisson's ratio does not apply to rods considered separately, that this ratio, therefore, has no application to a material like a slab of which rods form an integral part by their intimate union with the concrete, just as truly as if the reinforcement were made into a single sheet and combined with the concrete, although, of course, the value of K would not be identical in the two cases.

What the physical and mechanical properties of such a slab may be cannot well be predicted with certainty, although it may be pos-

Mr. Eddy. sible to give a rational explanation of what experiment may show its new and unforeseen properties to be. Whenever such a slab undergoes flexure which causes it to assume a dish-shaped deformation, the peculiar and unexpected properties appear; but it is just on this question of dish-shaped deformation in a slab that Mr. Godfrey has expressed views least in accordance with the facts.

Suppose, for example, a long building with two rows of columns parallel to the long sides, and panels nearly square, so that there are three tiers of panels lengthwise of the building, one long interior tier being between the two long wall tiers. Now, suppose heavy loads to be placed on three panels of a tier or bay extending across from wall to wall. If the writer understands Mr. Godfrey, he thinks that such an arrangement of loading would produce a critical stress in a mushroom slab much greater than would occur if the loading were removed from either two of the three panels. It might do this were it not for the wall supports and the stiff heads; but in fact the reverse is true. The walls entirely prevent the formation of the ends of any such cylindrical trough across the slab, and the stiff heads and walls will tend to accentuate the hollows parallel to the long walls, rather than those perpendicular to them.

The experimental test on which Mr. Godfrey relies to prove his contention has little or no bearing on the slab question, for similar reasons.

The panes of glass on which he experimented* had no continuous support at the edges, such as a slab has, to say nothing of additional local stiffness at the points of support furnishing at these points moments of resistance several times as great as those found elsewhere. The plates were not clamped to practically immovable supports so as to make them act in a manner at all similar to a floor slab such as that under discussion. Consequently, these plates would not act like a floor slab at all, and the assumption that they would is entirely misleading. As just stated, the behavior of glass plates supported in this manner would be the reverse of the slab, in that the troughs in the one would be accentuated in a direction different from those in the other.

This and other matters in the discussions connected with Mr. Nichols' paper directly controvert the statements of Mr. Godfrey, which he evidently regards as incontrovertible, as he has made these statements a number of times and they have never been controverted, as he says; and all this will occur without contravening any law of Nature.

All engaged in concrete design and construction should be equally desirous, with Mr. Godfrey, that it should be conservative and safe, and also that it be adopted where it can safely be done at a cost so

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* *Proceedings*, Am. Soc. C. E., for August, 1913, p. 1361.

moderate as to permit fire-proof in place of combustible construction, as it cannot be if there are unreasonable requirements. Mr. Eddy.

It is not lightly to be assumed that any concern which has been responsible for the design of about two thousand structures of this character without failure, and with inside opportunities for knowing with certainty what can be depended on, and having also enormous financial interests at stake, may not be a good judge of what can and ought to be done in this field.

Mr. Godfrey refers to a paper by the writer entitled "A Comparative Test of Two Full-sized Reinforced-Concrete Flat Slab Panels,"* and tries to convey the impression that the manner of loading and the design of the columns were responsible for the good showing made by the mushroom slab. He states that the test slab was loaded with balancing loads, meaning by that that the load on the over-hang beyond column centers reduced the stress in the central panels so greatly as to make the stresses in the steel small and misleading.

It will be noticed that Mr. Eckles asserts that his experiments showed greatly increased central deflections due to extending the loading into the panels at the sides of the panel tested. These contradictory views cannot both be true.

The truth is that neither of them represents the facts. If there is anything that has been established by numberless load tests of mushroom slabs, it is that in the standard mushroom slab the central deflection and stresses of an interior panel are not affected to any appreciable extent by the loads on surrounding panels. A very small effect has been thought to be discoverable at the centers of panels situated diagonally, but no mutual effect between panels beside each other.

This fact, which would at first sight seem inexplicable, is apparently the result of a close balancing of two opposing effects, *viz.*, any load just outside a panel will have a tendency to lift the center of the panel, but, at the same time, it will tend to increase the deflection of the side belt between it and the center of the panel. Any such deflection at the side belt will have a tendency to increase the deflection at the center of the panel. Whether the lifting or the increase of deflection preponderates depends on the relative rigidities of the belts and the slab and column heads, with the result stated above. Not having at disposal sufficient data in other types of construction to state conclusions with certainty, the writer has refrained from doing so.

The panel tested by Mr. Eckles was at a corner of a slab. It may have been a mushroom slab in general design without having any features of the system in this corner panel beyond one mushroom head at one of the four corners. More details are needed in order to show that the system is in any way responsible for the results of Mr. Eckles'

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* *Engineering News*, March 27th, 1913.

Mr. Eddy. test. The writer has not attempted to give an analysis for such a panel. It is customary to design wall panels differently from interior panels, but that is no argument against the approximate correctness of the analysis for interior panels.

To return from this digression to Mr. Godfrey's criticism that the results of the test slab were due to balancing loads on the over-hang: The tables and diagrams of the test show that in Loads 3, 4, 5, and 6—which were the ones particularly relied on for showing the behavior of the slab—the loading resting directly on the panel was more than twice as great as that on the entire surrounding over-hang, and, in the case of all the other loads, that on the panel itself was greatly in excess of that on the over-hang. These, consequently, were not balanced loads, and besides, there was no loading whatever on the areas above the column heads for Loads 1 to 6, inclusive, and no large loads later. The loads were applied over an area in the form of a Greek cross, with the intention that whatever load might be applied would be effective in causing bending moments and be situated so as to be less favorable to the mushroom slab than to its competitor.

The columns were 18 in. square and 5 ft. 6 in. from top of footing to bottom of slab, and not, as Mr. Godfrey asserts, "about half as thick as their height." Their diameters, as well as those of the mushroom heads, were in the usual relation to the span. This structure corresponded to a basement floor slab, but, because there were no upper floors, the columns necessarily had very small load stresses per square inch. That, however, did not affect the resistance the columns would afford to unbalanced moments, except to render the columns subject to actual tensile stresses in their reinforcing rods by reason of the small superimposed load. The columns were at a disadvantage for this reason and also because the unbalanced bending moment in the slab was not resisted by the stiffness of columns extending upward as well as downward from the slab, as would occur in a building. That may possibly be regarded as partly offset by the fact that the columns were only 5 ft. 6 in. long, but there was no such divergence between usual conditions and those of this test as Mr. Godfrey asserts.

Nevertheless, this test was not made (as seems to be assumed) to demonstrate the strength of this form of construction, *per se,* but to make a comparative test between it and another form of construction, and to demonstrate that though one operated on the principle of the beam the other did not.

The writer had nothing whatever to do with the design of the slab or the arrangement or carrying out of the test, except to suggest the Greek cross as the ground plan of the loading. The data obtained were submitted to him for discussion. Mr. Godfrey or any one else has the opportunity to discuss them and come to any conclusion the figures will support. Innumerable load tests of standard mushroom

slabs have been made. This test was not made to find out anything about the elastic behavior of mushroom slabs, about which there was any real doubt or lack of knowledge by those designing these structures, but to put the facts in evidence, and further to establish their manner of failure when stresses are carried beyond the yield point. This last was a most important piece of evidence which had not heretofore received such experimental certification as to settle it beyond dispute. Mr. Eddy.

It would seem on the face of it as if this test on a full-sized slab, in which all the details are carefully tabulated by reliable observers in such form as to admit of complete comparison with any other tests, might at least afford as good a foundation on which to base conclusions as the experiments made by Mr. Godfrey on a couple of glass plates under conditions not in accordance with those obtaining in floor slabs, and yet Mr. Godfrey intimates that absolute dependence is to be placed on the indications of his glass plates, and that the slab test should be disregarded as untrustworthy.

It may be that Mr. Godfrey believes that this test "has not even a theoretic interest", but he may find it difficult to persuade any to agree with him except those who are so wholly blinded by prejudice or interest as in fact to be unwilling that any reinforced concrete structures shall be built for use unless designed in such an innocuous manner as to be "out of the running."

Mr. Godfrey has also repeated his unsupported statements as to the unreliability of reinforced concrete construction under rolling or jarring loads, and seems to regard his mere assertion as sufficient to settle the question in this case also. In order to show how groundless is this opinion, which apparently has no other foundation than his own say so, the writer would quote from W. K. Hatt,* M. Am. Soc. C. E., as follows:

"It is well known that concrete, because of its lack of elasticity, absorbs or deadens vibrations, and the sound caused thereby. It is not probable that vibrations reach the steel. The speaker has knowledge of many experimental attempts to loosen the bond by shocks and vibrations. So far smooth bars encased in concrete that have been subjected to shocks and long-continued vibrations seem not to have lost any of their original strength of bond. Likewise the concrete on the compression side of a reinforced concrete beam that has been loaded and released from load some 2 500 times to high working stresses seems not to have been substantially weakened thereby."

For, far as these experiments go, they give no ground whatever for Mr. Godfrey's statement, into which he would seem to have been led by applying a line of reasoning derived from the behavior of all-steel structures which is inapplicable to reinforced concrete. An impact

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. III, 1907, p. 60.

Mr. Eddy. or blow at any point of a steel bridge is propagated longitudinally along elastic members extending linearly from the point, and it goes practically undiminished to the farther ends of those members, where it is subdivided among other members and propagated still farther. An allowance of 80 or 90% is usually added for impact to the static effect of a moving load.

Impact, or the effect of a blow at any point of a reinforced concrete slab, however, is entirely different from this. In the first place, the effect of the blow does not travel in one direction only, but in all directions radially from its point of application, so that in a very thin slab, its effect at any other point would be inversely as the distance, and in a very thick slab inversely as the square of the distance. This would make the allowance for impact in a slab very small.

Secondly, the effect of impact must be inversely proportional to the weight of the body receiving the blow. It may be assumed that steel costs from twenty to thirty times as much per ton as reinforced concrete, and the effect of impact on a steel structure costing the same as a slab would be from twenty to thirty times as much, for that reason.

Thirdly, the continuity and stiffness of the slab greatly reduce its vertical, lateral, and torsional deformations below those of a steel structure. The work done during impact, and its effect, depends on the amplitudes of the deformations. In particular, the horizontal resistance of a slab is many thousand times that of a steel structure. The vibratory energy absorbed by a slab during an impact is consequently small.

Fourthly, the small amount of energy which is absorbed is not transmitted (as it is in a highly elastic and resilient structure) to a considerable distance in the slab, but, owing to the nature of concrete, is dissipated near its source, transformed into heat, and rapidly absorbed.

Fifthly, slabs are tough, and not brittle, like terra cotta, for example, so that, in cases where great weights have fallen on them, little effect has been produced, whereas brittle slabs have been smashed under such circumstances, and have failed.

The concrete of the compression zone is such a shock-absorber as to protect the tension zone from jarring and vibration, both as regards steel in tension, and concrete as well.

For all these reasons the shock which a rolling load imparts to a slab is inconsiderable, and is absorbed and dissipated so readily that it is a negligible factor, instead of being such an important and overshadowing consideration (as Mr. Godfrey says) as to prevent its safe use in railroad viaducts and the like.

In support of these contentions the facts respecting the concrete railroad bridge, Fig. 21, which carries the track of the Soo Line between

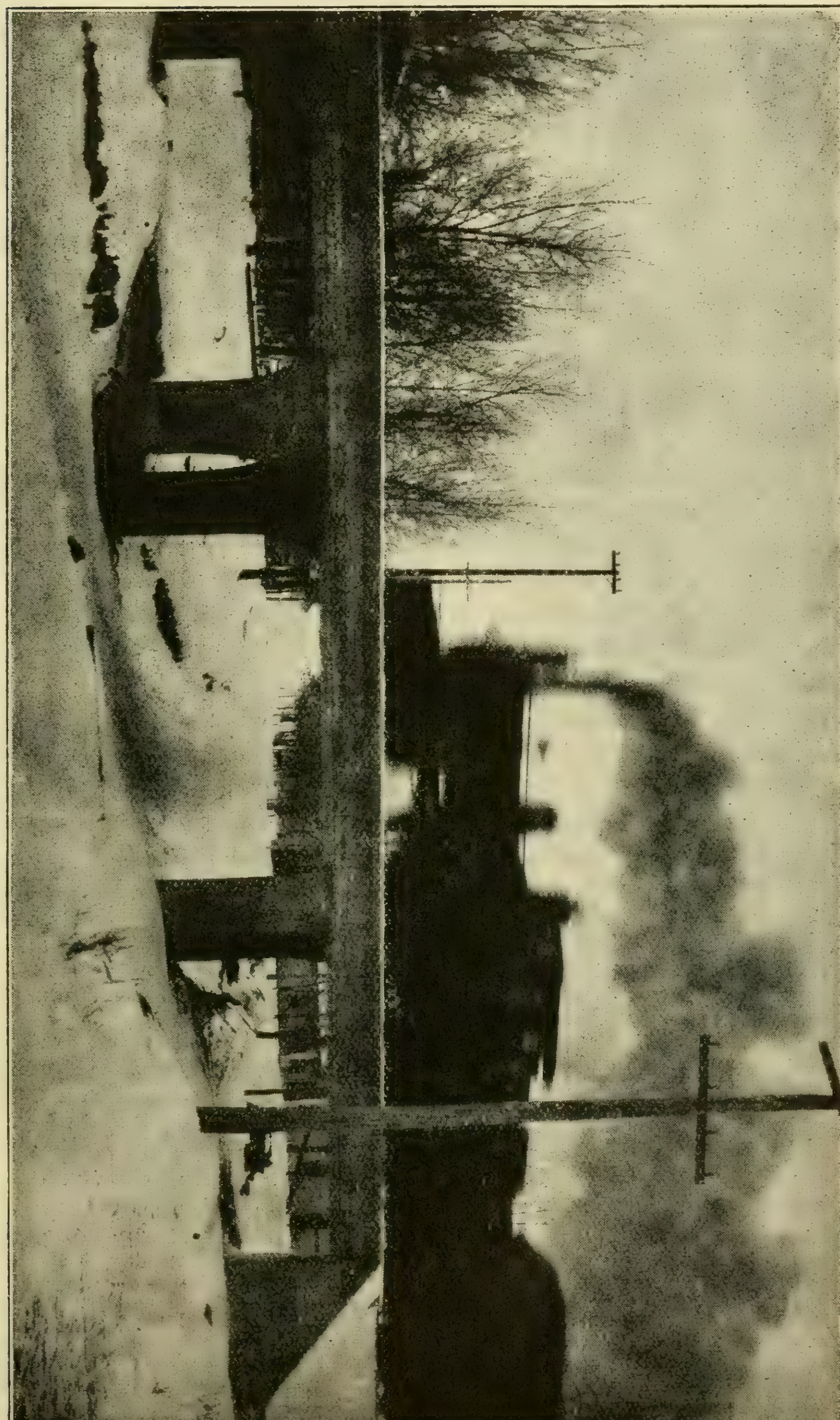

FIG. 21.—SOO LINE RAILROAD BRIDGE OVER WILSON STREET, AMHERST, WIS.

Chicago and Minneapolis over Wilson Street at Amherst, Wis., are instructive. It is a slab or girder, 18 ft. 6 in. wide and 2 ft. 3 in. thick, with reinforcement in excess of 1%, acting largely as a girder, but with some slab action, having a central span of 25 ft. 9 in. and designed to carry the heaviest locomotives with inappreciable deflections. It would be extremely difficult to cause failure by any loading that could be placed on it. It would require a load of more than 600 tons on this panel to cause a unit stress in the steel of 16 000 lb. This bridge has given such excellent satisfaction for years that the Superintendent of Bridges and Buildings of the Soo Line considers reinforced concrete as the desirable material for such spans. Mr. Eddy.

Particulars respecting other satisfactory slab bridges carrying street-railway traffic are to be found in the writer's book.*

It would seem that Mr. Godfrey's opinions respecting the availability of reinforced concrete for such purposes are not in accord with successful practice, and are not such as should be expressed with vehemence until they are supported by careful experimental tests, rather than by vague references to failures which are used merely to cast discredit on reinforced concrete design in general. There have been lamentable failures of all-steel structures, but that is no reason for discarding them. It is reason for discriminating analysis and careful design. The same lesson is to be derived from failures of reinforced concrete design, rather than to use them as a pretext for indiscriminate condemnation of reinforced concrete as unsafe for moving loads.

Mr. Eckles must have made some mistake in ascribing to the writer any previous attempts to treat reinforced concrete flat slabs. It is true that a paper on "Thin Homogeneous Circular Flat Plates" was published by the writer some years ago, but it contained no reference to any application of the flat slab theory. In the paucity of theoretical developments which could possibly furnish the basis for slab theory, this paper was used again and again in current engineering literature for that purpose, but the writer was in no way responsible for this, and in fact found so much in these applications with which he was at variance, for one reason or another, that he was induced to investigate the theory of reinforced flat slabs, as he has done in his book. The attempt now under consideration is his first effort in that line, and is entirely independent of his previous paper, and any reference to "his past efforts" simply reveals the lack of information of the one making such reference, and has no foundation in fact. When Mr. Eckles mentions discrepancies between the writer's former theories, and conclusions based on facts, it would be well to cite references, as the writer cannot recall any work of his to which these remarks apply. The only development of slab theory in any work of his is in the one on "Flat Floor Slabs," in 1913, and the paper now

* "The Theory of Flat Slabs," pp. 67-69.

Mr. Eddy. under discussion is the application of the theory there developed to test results in several slabs. There is certainly no discrepancy here, such as Mr. Eckles would have us believe.

Mr. Thompson apparently does not understand the object the writer had in view in his paper. It was to show that test results as to deflections and stresses, as far as they were available, were in such accord with the analysis which he had proposed in his book that a reasonably close practical agreement existed between them, especially in the case of the mushroom system with which he was in a position to obtain inside information more complete than was the case with other systems. At the same time he utilized for comparison such information respecting other systems as was at his disposal.

Mr. Thompson seems to think that if he could show that the test slab of the Northwestern Glass Company's Building was insufficiently reinforced, or over-strained by the test load, or otherwise incorrectly designed, he would thereby discredit the computations in the paper; but such is not the case.

It is no doubt the truth, as Mr. Thompson alleges, that it is possible to draw further conclusions from the test data given than were drawn by the writer, and it may be surmised that the four conclusions stated by Mr. Thompson at the beginning of his discussion do not by any means exhaust the list. It is assumed that wall panels must necessarily differ in design from interior panels, and that every one conversant with slab design is perfectly aware of this, and hardly needs to have it called to his attention. It is believed that wall panels invariably differ in this way. The amount of that difference, however, is dependent to a large extent on the character of their connection with the walls and pilasters.

The deflections of Panel A shown in Fig. 7, when compared with those in Panel D, do not indicate that in this case the moment of resistance of Panel A as actually designed needed to be increased by more than a slight percentage to make its resistance equal to that of Panel D. As this is a question not yet entirely amenable to analysis, it remains to a very considerable extent to be determined in each case by the judgment of the designer. Other types of slabs having more flexible connection with the columns would no doubt require greater additional stiffness in wall panels than was needed in this case.

The principal questions at issue, however, as seen by Mr. Thompson, appear to be based on his two assertions that Equations (*a*) and (*b*) do not in fact give results that accord with tests, and as the principal support of that assertion, the further assertion that the observed stresses at gauge line 111 were not abnormal, but such as to be expected. The writer will proceed to meet these assertions at some length and then treat other matters broached by Mr. Thompson.

It should be stated first that the writer had nothing whatever to do with the design or testing of this slab, nor with that of any of the other slabs treated in this paper, and it is a subject of regret with him that the number of gauge lines over the column heads was so small, and that they were so irregularly distributed. However, he was compelled to make the best use possible of the data at his disposal, and he is still of the opinion that his interpretation of the results is substantially correct, as he will proceed to show by making use of analogous data from an unpublished test which has been put at his disposal since the paper was written. Mr. Eddy.

Additional Data.—This is the case of a slab, 7 in. thick, with panels 18 ft. square, design load 150 lb. per sq. ft., and loaded at once over four panels forming a square, with a load which if uniformly distributed would have an intensity of 151 lb. per sq. ft., but disposed so as to leave several open spaces. It is estimated that the stresses in the steel over the center column were such as would be produced by a uniform load somewhat in excess of the design load. The four loaded panels were two wall panels and two adjacent interior panels, just as in the Glass Company test.

Fig. 22 is a plan of the central column head and the slab rods crossing it, with the outer ring rod 8 ft. in diameter, the inner ring rod 4 ft. 6 in. in diameter, and the cap 3 ft. 4 in. in diameter. The plan also shows the relative positions of the numerous 8-in. gauge lines, most of which were in the semi-circle most distant from the wall, and the length of the laps drawn to scale. Each belt consisted of thirteen ⅝-in. round rods, except the direct belts which cross the wall panels from pilaster to column, and these had sixteen rods each.

The measured stresses, with the mean embedment of the axis of each rod at the extremities of the gauge lines, are given in Table 16.

Gauge lines 14 and 44, were on the top and bottom of the same rod, as were 15 and 42 also, from which it is evident that the other observed stresses on the tops of rods are most certainly in excess of the mean stresses to which the rods were subjected. Especially must this be true at the edge of the cap, where there is a sudden change in the flexibility of the slab. For that reason it is quite uncertain what actual mean stresses are indicated by the stresses which were observed on the upper surface of the rods crossing the edge of the cap radially; but we may be sure that they are much smaller than those corresponding to observed elongations.

It thus appears that some part at least of the excessive stresses apparently found here are due to placing the gauge lines on the upper surface of the rods, and the explanation which has been offered for these excessive stresses as being due to the thick boss formed by the head is needed as a reason to account for only a fraction of the observed results.

Mr. Eddy.

Equations (a) and (b), proposed for computing the maximum stresses in the slab rods of side and diagonal belts, respectively, in the head, give results which do not differ greatly from each other.

Let $W = 50\,000$ lb., $L = 216$ in.
$A = 13 \times 0.11 + = 1.4365$ sq. in.
$d_3 = 5.5625$ in.

Then Equation (a) becomes

$$f_s = \frac{50\,000 \times 2 \times 216}{800 \times 5.56 \times 1.43}\left[\frac{3\,(216)^2}{(216 - 40)^2} - 1\right] = 12\,000 \text{ lb. per sq. in.;}$$

but $L'^2 = 2\,(216)^2 - (96)^2 = 102\,528$, and $B' = 265.4$, and Equation (b) becomes

$$f_s = \frac{50\,000 \times 216 \times 3.37}{400 \times 5.56 \times 1.43} = 11\,500 \text{ lb. per. sq. in.}$$

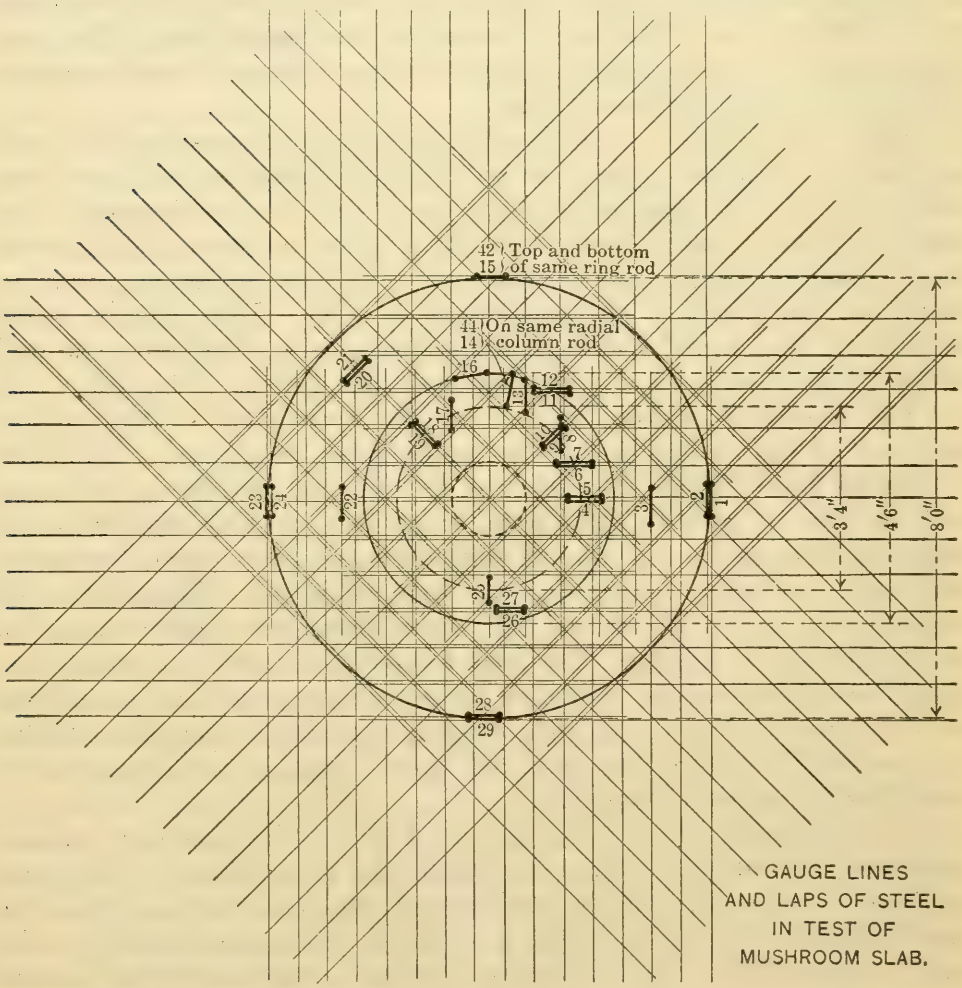

FIG. 22.

TABLE 16. Mr. Eddy.

Gauge line No.	Embedment, in sixteenths of an inch.	Observed unit stress, in pounds.
CENTER RODS OF BELTS.		
4	23	8 650
5	23	8 150
6	23	9 200
7	23	7 850
9	11	9 150
10	11	9 100
18	18	8 850
19	18	9 300
25	30	8 100
OTHER SLAB RODS.		
1	56	1 800
2	56	1 700
3	44	3 500
8	27	5 000
11	22	6 850
12	23	5 600
13	28	6 150
17	31	6 250
20	33	2 000
21	33	8 300
22	57	2 650
23	57	1 450
24	57	1 000
26	27	5 700
27	27	5 150
28	58	800
29	57	1 300
RADIAL AND RING RODS.		
14	48	3 350
44	......	2 300
15	57	1 850
42	......	550
16	33	3 750

The reason that these computed values are in excess of those observed, lies in the quantity of steel to be found in the laps, as shown in the plan, which quantity is evidently largely in excess of that for which the formula was computed, as will be seen on referring to page 32 of "The Theory of Flat Slabs," where it appears that the mean cross-section of the slab steel was assumed to be 4.25 times the area of a single belt. The case in hand certainly exceeds this, as all the belts are lapped as far as beyond the inner ring rod, with a consequent reduction of the stress at the edge of the cap of perhaps 25% below the value computed by Equations (a) or (b).

It should be noted that the observed results show some diminution of stress with the embedment, but, in the slab rods themselves, this is

Mr. Eddy. by no means proportionate at the edge of the cap to their approach to the neutral axis, although it is more closely proportionate elsewhere.

All these gauge lines were located around the central column corresponding to Column 45 of the Northwestern Glass Company Building. Besides these gauge lines, seventeen more were observed at three other column heads corresponding to Columns 36, 37, and 44 of Panel D, on the edge of the loaded area. The gauge lines, with one or two exceptions, were located (as were those around the central column) so that the edge of the cap nearly bisected each of them. The largest pair of stresses observed showed 7 400 and 7 700 lb. per sq. in., respectively, being on parallel adjacent diagonal laps. The next smaller observed stresses were 6 450 and 6 250 lb. per sq. in., respectively, with no other values as large as 6 000. In general, the values of the maximum stress in these gauge lines varied from 70 to 80% of those over the head of the central column. The two first mentioned appear to be abnormally large for some unknown reason as others similarly situated were much smaller.

Again, in Mr. Lord's report of the test of the Deere and Webber Building,* Fig. 14 shows cracks on the upper surface of the slab within an inch or so of the edge of the caps in the loaded area; but these cracks cannot be said to lie more outside than inside of the edges of the caps. It is clear, therefore, that those radial gauge lines that cross the edge of the cap in Fig. 22 will unquestionably show maximum steel stresses. Gauge lines 13, 14, and 44 give no indications that maximum stresses are to be found outside the cap. These additional data are thus seen to be confirmatory of the substantial accuracy and applicability of Equations (*a*) and (*b*).

Recalculation of Equations (*a*) *and* (*b*), *in the Northwestern Glass Company Building.*—Consider now the effect of the distribution of the actual loading. Suppose that Load 3, which consisted of 380 000 lb., were at first uniformly distributed over the four panels, each 16 by 17 ft. Next, let the loading on an area 7 by 8 ft. over the central column, and amounting to about 20 000 lb., be entirely removed and be piled on the surrounding parts of the slab. This will practically leave the cantilever portion of the slab around the column without loading, and will place this loading outside the inflection lines which surround this column. If similar removals occur at other columns, so that the loads are nearly symmetrically disposed on the central parts of the panels and the side belts, then it will be of little consequence, as far as the stresses in the cantilever portions of the slab are concerned, whether the loads outside the lines of inflection are uniformly distributed or have wide open ways across them; the stresses in the cantilevers will necessarily be increased by removal of the loading from the cantilevers to the other parts of the slab.

* *Proceedings*, Nat. Assoc. of Cement Users, Vol. VII, 1911.

Mr. Thompson says that the stresses "are more affected by the [total] amount of load in the panel than by their position". That certainly is not the fact, for, to take an extreme case, the loads might possibly be applied directly to the tops of the columns and produce no stress in the slab. The fact is that removal across the lines of inflection affects the stresses in the cantilevers, and removals within other portions of the panels affect the moments in those portions of the panels where the removals occur, but as long as they are symmetrical, such removals have little or no effect on the cantilevers. Mr. Eddy.

In the paper it is in effect assumed that this removal added more than 10% to the effect of the load, as far as the stresses in the steel at the edge of the cap of the center column are concerned, and such allowance is believed to be not too great. Mr. Thompson disagrees with this on general reasoning, based apparently on principles applying to beams simply resting on supports, instead of firmly fixed to stiff supports, in which the position of the lines of inflection would be affected by the distribution of the load to an extent which is not the case in slabs, where the lines of inflection are practically fixed in position and remain the same for different distributions of load. Mr. Thompson, however, apparently states the truth inadvertently when he asserts that "the same load uniformly distributed would not have caused larger stresses than was caused by the load placed as shown in Fig. 5", for, as we have just seen, it would certainly have produced smaller stresses.

In case the lines of inflection are practically unaffected by shifting the load from the cantilever, as just described, the stresses at the edge of the cap will be increased, and it will also be justifiable to neglect the effect of any change in distribution of loads due to the center aisles, 18 in. wide, when treating the cantilevers, though perhaps not in treating other portions of the slab.

On repeating the calculation of the maximum stress at the edge of the cap, as given in the paper, the figures were found to be somewhat inaccurate by reason of using an incorrect value for the clear span between the caps. The correct figures are:

Diameter of cap = 50 in.; diameter of head = 87 in.

$L_1 = 204$ in., $L_2 = 192$ in., $B_1 = 204 - 50 = 154$ in.

$L'^2 = (204)^2 + (192)^2 + (50)^2 = 86\,049.$

$L^2 = (204)^2 + (192)^2 = (280)^2$. $B' = 280 - 50 = 230$ in.

If $W = 106\,250$, $d_3 = 6.5$ in., and $A_1 = 15 \times 11 + = 1.6567$ sq. in.

Then, by Equation (a), $f_s = 20\,700$;

and by Equation (b), $f_s = 19\,500$;

which values may need to be increased somewhat for lack of laps, as previously stated.

Therefore, a mean value of 20 000, say, must replace the erroneous value, 17 000, originally given.

Mr. Eddy. It would be interesting to see the calculation by which Mr. Thompson says a value of $f_s = 15\,400$ is obtained by the application of Equation (*b*). The only change which he has suggested in the computation consists in scaling down the load of 106 250 lb. per panel, which he says is too large, though that does not appear to be the fact, as has already been shown. However, proceeding on that basis for the moment, let the load be scaled down to 96 000 lb. per panel, then, provided the originally computed value given in the paper ($f_s = 17\,000$ lb.) were correct, we should find $f_s = 15\,400$ lb. Recalculation, however, makes the value of f_s at least 19 500 instead of 17 000 lb., and the scaled value would consequently be 17 600 instead of 15 400, a discrepancy of 2 200 lb. per sq. in., or more than 14 per cent. It must be that Mr. Thompson did not actually compute this by the formula, but accepted without investigation the erroneous figure in the text.

There were, in fact, no laps whatever in the slab steel over Column 45, and the general formula, as discussed on page 32 of "The Theory of Flat Slabs," would need to be modified slightly for that reason, so that, in applying Equations (*a*) and (*b*) to this case, the reduction of the cross-section of the steel in the belts below the mean value assumed in establishing the formula would require a corresponding increase of the maximum stress at the utmost of, possibly, 20% above that computed from the tentative coefficients in Equations (*a*) and (*b*). Indeed, 10% in addition to the computed stress is sufficient to cover the abnormal unit stress of 22 000 lb. at gauge line 111.

It may truthfully be said, therefore, that there is good agreement of Equations (*a*) and (*b*) with stresses in this test, even if the actual stresses be assumed to be considerably in excess of any observed.

Abnormal Stresses at Gauge Line 111.—Mr. Thompson has misunderstood the facts stated in the paper respecting the stress at gauge line 111, as he has confused the elbow rods with the slab rods. The eight radial elbow rods, shown in Fig. 15 under the slab rods, are of $1\frac{1}{8}$-in. round steel, and are of great stiffness compared with the slab rods. As stiffness is proportional to the fourth power of the diameter, one of these rods is more than 80 times as stiff as a slab rod. These are the rods that were forcibly pried and held in position in the slab until relieved by the application of the test load, when they acted with that load in inducing stresses in the slab rods which they were unable to produce while acting alone, being held by the shrinkage stresses of the concrete. Thus, after the slab had once suffered flexure by the application of a considerable load, and had had a chance to recover, the radial rods and slab rods would come back to a position of equilibrium and mutual adjustment different from the one they occupied before the application of the load. A renewed application of loading would then show elongations and stresses measured from this new position

of equilibrium corresponding to the loading. If concrete was a perfectly elastic material, however, such an action would not occur. If any such steel stresses were around the edge of the cap at Column 45, as Mr. Thompson states, they could not fail to show at such a gauge line as 107, which crosses the edge of the cap. The assertion that the stress at 111 was not abnormal because it increased and decreased with the load, is inconclusive, because that depends on how much decrease occurred on the removal of the load as compared with the previous increase. The fact is that the decrease of stress at 111 on the removal of the load was so small as to make it apparent that some such action was occurring as has been described. That this is the correct explanation of the observed abnormal stress at gauge line 111 seems to be established conclusively by the observed unit stress of 11 600 lb. remaining after the removal from Panel C of all but 7 000 lb. of the load of 47 100 lb., and an observed unit stress of 10 300 lb. still remaining a day later. This abnormal stress after the removal of the load, such as is not shown at any other gauge line, makes it clear that the stress at 111 was abnormal before the load was removed, and nullifies any reasoning that leaves that fact out of account. Mr. Eddy.

Location of Gauge Lines.—A misconception is likely to arise respecting the matter of stresses inside the cap and outside of it, though disregard of the fact that absolutely sudden changes of stress cannot occur at any point of a rod, because such changes must be more or less gradual from point to point. Owing to this fact, and to the fact that the edge of the cap is comparatively thin, large stresses are to be found for some small distance inside the geometrical outline of the cap, and are not confined to the area strictly outside of it. Indeed, some designers, in order to take account of this, have assumed the effective diameter of the cap as 2 or 3 in. less than its actual diameter. This question has its bearing on the location of the gauge lines with reference to the geometrical edge of the cap.

Again, the single abnormal result cited by Mr. Thompson, and shown in his gauge lines, *A* and *B*, does not accord at all with what is generally known respecting the distribution of stresses at the edge of the cap, and, until more specific information is available, it should be disregarded because a single measurement of this kind cannot be depended on, even were it not known that it is abnormal. It would be quite understandable that whichever of Mr. Thompson's gauge lines, *A* or *B*, lay across a crack would show the greater stress. As seen in the Deere and Webber test, however, that might as probably occur with *A* as with *B*; or it may merely be due to a so-called kink in the rod being straightened out, or merely the extra tension in the upper surface of the rod.

Mr. Thompson's attack on the validity and general accuracy of Equations (*a*) and (*b*) is thus seen to have a very slight foundation in

Mr. Eddy. test data, and his estimate that the actual stresses may exceed those computed by these equations by as much as 45% may be disregarded.

Mr. Thompson says, respecting Mr. Turner's test slab, that it was broken by loads placed on the center panel, with comparatively little load on the projections. That is the truth, Mr. Godfrey and his balanced loads, to the contrary, notwithstanding. Mr. Thompson criticizes the test because it was not directed toward elucidating the particular problem in which he is interested. Mr. Turner, however, had in view the solution of a different problem, for which he was willing to expend money. Neither was that problem the one Mr. Godfrey would like to have seen investigated. It will be interesting and valuable for these gentlemen to investigate the problems they have in view, but they can hardly expect others to put aside their own pressing problems because they themselves are not interested in them.

It may be stated again, however, that, so far as can be learned from numerous load tests of mushroom slabs, central deflections of panels are unaffected by loading adjacent panels, which, if true, may permit this test to be used to solve some of the problems proposed by these critics. However, it is perfectly evident, from the report of the test and the photographs of the failure,* that Mr. Thompson is mistaken if he supposes that the steel and concrete just at the edge of the cap were ultimately more severely stressed than at points somewhat farther from the center of the columns, for the test slab gave way at points at least as far distant from the cap as gauge line 103, instead of at the edge of the cap, and this was a case of belts without laps.

Mr. Thompson entirely misses the point of the argument, respecting the comparison of the stresses at Column 36, and those at Column 46, which are similarly situated with respect to Load 3. At 36 the stress lines given are those largely due to the loads in Panel C, and at 46 to loads in Panel D. Whatever changes occurred at 46 by increasing the load on D might be expected to be paralleled, were the load on C increased. What is there "absolutely misleading" about this? From the behavior at 46 when the load on D was doubled, it might perhaps be surmised what would happen at 36 were the load doubled on C, which it was not. What misleading conclusion has been here advanced?

Certainly there is nothing so misleading in this as in Mr. Thompson's statement that the slab bar on which gauge line 103 was taken "was one of the outside ones of the diagonal band". In fact, however, the bar was the third one from the center bar and the outside ones were the seventh from the center bar, all equally spaced, and so this bar was not half way to the outer bar, where it was asserted to be by Mr. Thompson. That, unquestionably, is "absolutely misleading".

* "The Theory of Flat Slabs," p. 92.

Mr. Eddy.

Mr. Thompson criticizes the values obtained for stresses in the steel in the diagonal and rectangular bands because, he says they "are calculated from formulas derived without regard to the size of the column head, and, * * * would apply to a construction with a column head equal to a sharp point and one of any size whatever." He says "this assumption, on the face of it, is erroneous". Whether erroneous or not, one thing is certain, *viz.*, the stresses, if correctly computed on the assumption stated, will not become larger on account of making the head of appreciable size. The effect of the head, if anything, will be to decrease the calculated stresses at mid-span, and the error, if such it be, will be on the side of safety, and on that account is unobjectionable, from a practical standpoint, however much it may detract from the accuracy of the formulas. As the criticism is based on what appears "on the face of it", it might be well for Mr. Thompson to make an estimate, if he can do so, as to the amount of the effect produced at mid-span of these belts by ordinary sized heads, when due consideration is given to the fact of the comparative fixity of the lines of inflection, thereby conferring on the central parts of the panels outside the cantilever a semi-independent character, which justifies the assumption as one approximately correct. The final proof of the applicability of the formulas, however, is to be found in their accordance with test results, as has been shown in the discussion of the tests.

Mr. Thompson criticizes Equations (*a*) and (*b*) because the observed stresses at the edge of belts opposite column centers fall short of those computed from these equations. It must be remembered that these equations were derived on the supposition that the steel was as near the top of the slab as it is over the column caps. As it is in fact at a much lower level at the edges of the belts, less stresses will be observed here. This is part of the reason for the occurrence of large stresses at the edge of the cap; but the theoretical stresses thus computed furnish a basis on which to compute maximum stresses in slab steel over the head.

Mr. Thompson objects to the word "theoretical" in this connection. The word correctly designates a result reached from a consideration of the predominating principles involved. It does not mean, when applied to engineering structures, that all factors have been introduced into the premises. It means such simplified assumptions and premises as will take account of the dominating factors and give as a result a reasonable approximation which is accurate enough for practical purposes. As neither Mr. Thompson nor, so far as is known, any one else has proposed an approximate theory in any practical agreement with experiment, it would seem to be the courteous thing not to assail a proposed theory on the basis of assertions which seem not to be well founded.

Mr. Thompson says that a design, such as that of the Northwestern Glass Company Building, does not have a proper factor of safety, the

Mr. Eddy. inference being that radical changes of design are required, whereas it appears from the preceding analysis and tests that all that is required to bring down the unit stresses in the steel at the edge of the cap, and reduce them by any desired amount, is merely to place additional short rods across the cap between the slab rods, of a length from three-tenths to four-tenths of the span, and no general alteration of design is required, as is intimated.

Mr. Thompson refers at the same time to stresses in the concrete. It is not the object of the paper to go into that subject further than absolutely necessary, and it will be postponed for future consideration, with the remark in passing that, where an absolutely flat slab is not required, every one is aware that increased thickness of slab in the vicinity of the cap will reduce the concrete stresses.

What the writer is especially concerned about in this discussion is, not the close accuracy of the particular computations of this paper (for they are admittedly based on average data which do not necessarily coincide altogether with the particular examples to which they are applied), but to convince those interested in slab construction of the fact that a method has been found for making a rational analysis of slab design, a method which will ultimately enable the engineer to predict results with somewhat the same certainty as is now attained in ordinary structural work. That may require modifications of constants, etc., in his practical formulas, which apply to the mushroom system. Such modifications will be no greater than every engineer uses in bridge construction where a method is used, which is not expected to result in the same final formulas for stresses in all types of bridges.

This paper proposed to discuss "Steel Stresses in Flat Slabs," and see how closely they could be computed by the analysis published by the writer in his book. Mr. Mensch regards the paper as taking up the general defence of the mushroom system. That is an unwarranted enlargement of the scope of the paper. He seems to think that if he can make it appear that any of his numerous objections to flat slabs are admissible, he has thereby cast discredit on the paper in some way. Let it then be said, by way of preface, that Mr. Mensch has not called attention to a single discrepancy between the computed and the observed results. The only discrepancies mentioned by him are between results given and confirmed both by analysis and experiment with what he believes they should be.

The real question at issue, however, is this: can the writer actually compute steel stresses in any one type of flat slabs correctly? We are led to believe that Mr. Mensch thinks he cannot do this, but instead of meeting the main issue directly he introduces a number of side issues. The side issue with which he begins and ends his remarks is the question of the factor of safety in flat slab construction, and the load tests he has made on his own constructions, showing a factor of safety of at

Mr. Eddy.

least 4. He also "claims that flat slabs designed according to Mr. Eddy's formula have only a factor of safety of 2, under the most favorable circumstances". Now, this is not so, because, in the case of the test of the St. Paul Bread Company Building, given in the paper, and designed according to the standards of the mushroom system for a load of 100 lb. per sq. ft., one panel was loaded with 415 lb. per sq. ft., without showing signs of distress or any cracks except a few very fine hair line cracks, to be discerned only with difficulty. That observed result would seem to have more weight than any mere estimates on which Mr. Mensch bases his claims, and would tend strongly to make us put no trust whatever in the claims which he has put forth so confidently. According to him, that slab ought to have failed under half the final test load. The truth of the matter is that the materials of which it is composed are used more economically and will carry more load than if disposed as Mr. Mensch proposes in a slab-girder construction, and though he is willing to concede a slight advantage to flat slab construction, weight for weight, his adoption of a wrong theory prevents him from giving flat slabs the credit to which they are entitled.

Mr. Mensch declares that extensometer measurements of steel stresses afford no correct indication of the strength of a slab, and load tests, evidently, in his estimation, afford no reliable indication of what may be expected beyond the loads actually applied, as he believes a slab is subject to sudden collapse from sudden and unforeseen large stresses in the steel which may be expected to develop with slight increments of load; but, were this the case, this phenomenon should have been met frequently in the course of the numerous tests which have been made. So far from this being the case, the fact is that multiple reinforcement is of such a nature that unusual stresses developed accidentally in any rod of a slab must ultimately be distributed and divided among neighboring rods and thus be safely carried.

Mr. Mensch is mistaken in ascribing the discrepancies between observations at symmetrical points in slabs to sudden changes in stress under comparatively small increments of loading, such as have occurred in beams. A study of the stresses given in connection with the test of the St. Paul Bread Company Building, where these discrepancies were observed, fails entirely to reveal any such phenomenon as this. The discrepancies were in general as pronounced at smaller as at larger loads, with no such sudden increases of stress as alleged. This might well be expected in beams when cracks develop, but a crack such as ordinarily occurs in a slab would not have any such observable effect. It would be necessary for cracks to cross one another in the slab in a way such as they practically never do, to permit even the initiation of effects similar in character to those due to single cracks in a beam.

Almost at the beginning of his discussion Mr. Mensch turns from the discussion of slabs to that of beams, which he makes his principal

Mr. Eddy. side issue. He says that "one may reasonably expect that the discrepancies will be still greater in flat slab construction", because the percentage of reinforcement is less than in beams. The entire purport and effect of the reasoning and the results reached in the paper go to establish the fact that slab action and beam action are different, theoretically and numerically, and it seems almost useless to attempt to reply to criticisms based on a complete confusion of beam action with slab action. Mr. Mensch says that slab action is "simply a combination of continuous beams acting in at least two directions, which beams are connected with each other and with the columns, and the deflection in any point is common to at least two beams acting at right angles to each other." By that statement he means to say that a theory of computation based on such an assumed network of beams combined together would constitute a sufficiently close approximation in its action to that of a flat slab to afford a practical basis for obtaining the stresses in a flat slab.

The writer does not hesitate to deny that there is any such agreement between the results of such a theory and any set of trustworthy experimental observations that Mr. Mensch can produce. Mr. Mensch refers in particular to Danusso as an exponent of this correct theory, and as one who expresses real slab action in a scientific way. Danusso's papers, to which he refers, have been translated into German and published in book form, so that they are perfectly accessible to any one.* As must be evident, such a network of beams necessarily has supporting girders at the edge of each panel, and the system corresponds to a slab-beam-girder construction or else to a slab-girder construction and not to a flat slab construction. Danusso, consequently, makes certain suppositions respecting the stiffness of these supporting girders, as is perfectly proper to do.

The writer has developed a slab-girder theory which is awaiting more complete experimental confirmation before publication, but any such theory must necessarily differ from flat slab theory and it is absurd on the face of it for any one to think that the results of either theory are applicable to another construction from that contemplated in deriving it. Flat slabs without supporting girders are alone under discussion in this paper, and Mr. Mensch's definition of slab action quoted above does not apply to actual flat slab action.

Mr. Mensch says that "the advocates of many flat slab systems declare that the common beam theory does not apply to flat slabs," and that he "has never seen a similar statement in the works of St. Venant, Winkler, Grashof, Föppl, or Müller-Breslau," and that, "in particular, the case of flat slabs was solved by Winkler and Grashof by that theory". Of course, St. Venant, Winkler, and Grashof made no

* "Kreutzweise Bewehrte Eisenbetonplatten," Danusso-von Bronneck, Berlin, 1913, Wilhelm Ernst und Sohn.

statement about flat slabs, for the very good reason that they had not been introduced in their day. Flat plates, however, is a different question. A flat plate is understood to be homogeneous and of the same moment of resistance throughout; a slab is neither of these, but has increased moment of resistance where required locally. St. Venant, the greatest investigator of the theory of elasticity that ever lived, was much busied with the theory of flat plates thick and thin. A full and critical account of his researches may be found in the encyclopedic "History of the Theory of Elasticity and Strength of Materials," by Todhunter and Pearson, occupying pages 833 to 896 of Vol. I, and pages 1 to 282 of Vol. II. He did not treat flat plates on the beam theory at all, but on the basis of the exact general theories of elasticity, without regard to the approximations involved in the beam theory. He was born in 1797 and died in 1886. Grashof, born in 1826, published the second edition of his great book on "Elasticity and Strength" in 1878. This is the edition to which Mr. Mensch has referred elsewhere and is the one always quoted. It contains his investigation of homogeneous flat plates, but nothing about slabs. Those who have not ready access to this work will find the substance of the more important parts of his plate theory in Lanza's "Mechanics." Mr. Eddy.

It is impossible to explain how a man of Mr. Mensch's careful scholarship could have been led to say that Grashof solved flat plates, not to say flat slabs, by the beam theory. Nothing could be farther from the truth. Grashof is regarded as the great exponent of the slab theory, if not the originator of it, as distinguished from beam theory. It is true that Mr. Mensch in a paper of his* has used Grashof's formulas, and may have supposed that they were established as beam formulas, but such is not the case. Müller-Breslau has not treated flat plates or slabs in any of his published works. It is useless to try to have any beam theory of flat slabs accepted on the authority of any of these writers.

Danusso, in his preface, quotes at first the formula of "Grashof's plate theory", as he calls it, and later the Bach-Föppl formula, which is a convenient modification of Grashof's. It thus appears that Föppl used the plate theory rather than the beam theory that Danusso uses.

It is entirely gratuitous for Mr. Mensch to charge up any defects in the Niagara Building, to which he refers, to others than those responsible for its design, and it may be that the writer will be as ready as any one to condemn its faults if such there be, when he is apprised of the details of the design, of which he is as yet ignorant.

Mr. Mensch criticizes the idea of saying that a set of loads extending across a slab would tend to produce a cylindrical trough or depression under it across the slab, and that another set of loads extending across the slab at right angles to the first set will also tend to produce

* *Proceedings*, Nat. Assoc. of Cement Users, 1911, p. 205.

Mr. Eddy. a cylindrical hollow or trough across the slab under it, so that in the panel where these troughs cross each other the slab will be "dished" and their mutual effect will be for each trough to diminish the other. He declares that this description is "mysterious" and one that "will not enlighten any one", and yet this is the very idea of a cylindrical trough that Mr. Godfrey uses as the basis of his argument in his experiment. The idea is used by the writer merely as a semi-popular form of description, to assist the imagination in picturing this phenomenon, a phenomenon which he has investigated analytically from an entirely different point of view. It was used in order to put forth a form of explanation which would enable the reader to see how the results which had been reached by his analysis might possibly be pictured; and yet Mr. Mensch says this explanation is used because the writer does not care to express real slab action in a scientific way. It is evident that the critic has not followed the analysis, and is not even aware that that is exactly what has been done in the writer's book, where he has based his analysis on the general equations of equilibrium of the forces, shears, and moments acting on the external surface of an infinitesimal element of the slab, as is necessary in order to make a perfectly exact and rigorously scientific investigation of the slab.

It is evident that, with such divergent views of what constitutes slab action and slab theory, it would be useless for the writer to try to come to any understanding with Mr. Mensch within limits now at his disposal, especially when he insists, as he does, on the theoretical identity of flat slab construction with girder and slab construction.

In the interest of truth and fair play there is one correction that should be made in Table 15 given by Mr. Mensch. This table, which has been quoted several times in current literature, is based on one originally given by Mr. A. B. MacMillan, in which he computed the steel in the Turner slab for two different values of unit stress, *viz.:*

(*a*) at unit stress 16 000; weight = 549 lb. per panel;
(*b*) at unit stress 13 000; weight = 718 lb. per panel.

Mr. Turner explicitly stated that his formula, $\frac{W\,L}{50}$, was based on a unit stress of 13 000 lb., and Mr. MacMillan* made the same statement. The 718 lb. is consequently the only figure for which Mr. Turner would be responsible, and the weight of 549 lb. is entirely unauthorized, as his formula of $\frac{W\,L}{50}$ avowedly was based on a unit stress in the steel of 13 000 lb., as appears by reference to his book, page 29. For an assumed unit working stress of 16 000 lb., Mr. Turner's moment formula would become, in round numbers, $\frac{W\,L}{40}$.

* *Proceedings*, Nat. Assoc. Cement Users, Vol. VI, p. 266.

Mr. MacMillan further stated his ignorance respecting the amount of steel in the radial and ring rods of Turner's mushroom head, and assumed that their function was merely to support the slab rods in position at the top of the slab. The steel in the head would, in fact, be in the neighborhood of 200 lb., and it is evidently sufficient to add considerable stiffness over the columns, so that a conservative estimate of the steel per panel in Mr. Turner's construction would be 900 lb., instead of the figure wrongly stated by Mr. Mensch. This figure would be further increased by the usual laps over the column heads, so that substantial misrepresentation is made, intentional or not, by this garbled quotation. Mr. Mensch was himself present and contributed to the discussion of Mr. MacMillan's paper and has first-hand knowledge of these facts. It should be further noted that Mr. MacMillan's design, as shown by him in his paper, is almost identical with Mr. Turner's, with the omission of the inner ring rod, and his weight of steel only slightly exceeds Mr. Turner's design, as above corrected. Mr. Eddy.

He states* that:

"Floors designed by his method have been low in cost, have shown remarkable powers of resisting abuse, have stood tests of twice the live load over an entire bay, not only without signs of failure, but with such trivial deflection that one is led to believe that the constituent materials are far from being stressed inordinately."

Observe that this loading is not over one panel only but "over an entire bay," which disposes of Mr. Godfrey's contention that such loading would be fatal to this kind of structure.

It will be noticed that Mr. MacMillan's computation of the slab steel, according to Grashof's formula, provides nearly the same weight of steel as Mr. Turner uses, and would properly require additional steel to form a stiff head such as Messrs. Turner and MacMillan use. In round numbers, the weight of steel in either of these three slabs is, in all, 1 000 lb. per panel. These computations are really based on a flat plate theory of slab action in distinction from beam theory. The other four computed weights in the table are really based on beam theory, and require about twice as much steel, or about 2 000 lb. per panel. Beam theory practically neglects the co-action of the various belts, which co-action is considered and allowed for in slab theory. It affords a partial relief from the tensile stresses arising from the applied moments, a relief traceable to action of the bond shear of the embedment.

The statements that have frequently been made respecting the meaning of the results shown in this table seem to be misleading, as a correct interpretation of them is apparently to the effect that a beam arrangement, such as is involved for example in two-way rein-

* *Loc. cit.*, p. 266.

Mr. Eddy. forcement, requires perhaps twice as much steel as a slab which can develop true plate action.

The advocates of beam theory insist that any flat plate action that may be developed is uncertain and unreliable, and that slabs, in order to be safe, must be designed by beam theory.

The advocates of flat plate theory insist that innumerable tests have shown that flat plate action affords a perfectly reliable and dependable form of resistance when the design is properly made. The evidence for this proposition is regarded by them as perfectly conclusive, and they find it difficult, to say the least, to endure with patience the intimations which are continually made that flat slab design is essentialy unsafe unless it fulfills the requirements of beam theory. It would be just as logical to insist that a dome constructed so as to resist circumferential tensions must be designed on arch theory alone, and neglect circumferential action in the dome, as to insist that a flat slab, with a multiple-way reinforcement tied together by the embedment, must neglect the effect of that embedment.

Mr. Hatt is ready freely to admit the facts of which all who have made tests on buildings are perfectly aware, namely, that the observed stresses in the steel are actually about one-half as great as would be found by considering the steel to resist the entire applied moments.

If other critics have known the facts and have not been willing to acknowledge them, they have been disingenuous; if they have not known them, they were disqualified to discuss the subject by their ignorance of the essential basis of the discussion. Either alternative would seem to discount the value of the opinions which such critics have expressed, and would seem to make the basis of their opinions dubious.

When Mr. Hatt, however, refers to the tensile stresses in the concrete, which he says carry the remaining fraction of the applied forces, he evidently has in mind direct tensile stresses in concrete, acting parallel to the steel and to be added to those in the steel, such as would disappear and be ineffective whenever cracks appeared across these lines of stress. That is a naïve and quite natural explanation, and one which is evidently correct in the case of reinforced concrete beams, but can be shown to be incorrect in the case of slabs.

Owing to the stresses which take place in multiple-way reinforcement in slabs, the shearing stresses due to the bond between reinforcement and concrete play a rôle, and have a fundamental importance, that has not hitherto been realized or understood, and they produce effects such as they do not produce in beams. This has been explained elsewhere. Concrete stresses arising from bond shear may be correctly designated as indirect stresses, in distinction from those previously

mentioned as directly due to bending. They are operative and to be depended on as long as the bond remains intact, which is until final failure, and long after the cracks due to the direct stresses of bending have appeared. These indirect stresses are the ones which produce the effects Mr. Hatt recognizes. His judgment tells him that direct tensile stresses are unreliable. Agreed; but should he become convinced, however, that the concrete in combination with the steel in the slab affords a resistance of a reliable kind, as it does not in beams, he would be in a position to revise his estimates as to a correct basis for safety of design. Mr. Eddy.

In tests up to perhaps one-half of the design load it is usual for direct stresses in concrete to be of so much assistance to the reinforcement as to make the steel stresses comparatively small. For larger loads, up to two or three times the design load, or more, direct stresses cannot be relied on, but indirect stresses are predominant in the concrete. The true properties of reinforced concrete, as a combination which is sufficiently fine-grained to exhibit resistance such as might properly be ascribed to a single homogeneous material different from either steel or concrete, then appear. It is to this that the formulas of the book on Flat Slabs apply, as they do not to lower stresses. The material is one that is tough and not subject to sudden collapse or unexpected failure, and will show perfectly evident signs of distress long before gradually giving way. Such material must not be made with its bars too large or too far apart, as that makes it too coarse-grained and decreases the surface on which bond shear acts.

Comparatively few tests to destruction have been made in slabs, but those that have been made fully support the view here advanced. When, however, the experimental evidence shall become sufficient, Mr. Hatt will feel compelled, doubtless, to accept it as a fact, but the same can hardly be said of a number of those who constantly discuss slab theory in print, but who are so wedded to preconceived *a priori* conceptions as to how slabs must act as to be wholly incapacitated for any change of view, no matter what the facts may be. For reasons stated at the beginning of the paper, the writer disagrees with Mr. Hatt's opinion that the controlling factor in slab design should be the compressive stresses in the concrete rather than the tensile stresses in the steel. One reason for this is the uncertainty still existing as to the resistance which concrete or any other solid can offer to hydraulic compression. For example, does any engineer suppose for an instant that the rocks in the depths of the earth's crust, which are subject to enormous hydraulic stresses by reason of the superincumbent strata, are thereby crushed or rendered less able to resist shear or tension?

Mr. Eddy.

The condition of the concrete under compression in a slab is not comparable to that in a test block or cylinder in a compression test, so that the uncertainties respecting the concrete in a slab are greater, if possible, than those respecting the steel, and the measured compressive stresses in the concrete, on which Mr. Hatt would rely, furnish no reliable criterion, for it is highly probable that shearing rather than compressive stresses are the determining factors in the ultimate resistance of the concrete.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ROAD CONSTRUCTION AND MAINTENANCE.

An Informal Discussion.*

By Messrs. William de H. Washington, Willis Whited,† William H. Connell,† P. E. Green,† A. F. Masury, and W. R. Farrington.†

ENGINEERING ORGANIZATIONS FOR HIGHWAY WORK.

William de H. Washington, Assoc. M. Am. Soc. C. E.—In connection with the discussion of the subject of highway engineering organizations, the members of the Society are referred to a report to the Hon. John N. Carlisle, Commissioner of Highways of the State of New York, by the Board of Consulting Engineers consisting of Harold Parker and George C. Diehl, Members, Am. Soc. C. E., and the speaker. In addition to covering specifications for various types of roads and pavements and the materials to be used therein, the report considers the subject of efficient organization for the State Highway Department of New York, and gives in detail the work to be covered by such an organization, the duties of the various engineers included in the organization, and charts covering the interrelationship of its component parts. The report is on file in the Library of the Society. Mr. Washington.

Willis Whited, M. Am. Soc. C. E. (by letter).—In reply to the very able and interesting discussion which has been elicited on this subject, the writer would call attention to the following points: Mr. Whited.

Mr. James includes Pennsylvania among the States in which road taxes are paid in labor. This was abolished by an Act of Assembly in 1911, and all road taxes are now paid in money.

Much is said on the subject of the construction and maintenance of improved roads, that is, roads covered with some kind of paving. More than 90% of the roads in the United States are not improved, in

* Continued from March, 1914, *Proceedings*.

† Closing discussions.

Mr. Whited. the sense mentioned, and probably will not be for many years to come. The most important problem that confronts the State highway administration, is to make these roads as good as possible at a minimum expense, and generally with a view to more permanent improvement at some future time. If an engineer has work to do that is worth \$10 000, and has \$10 000 with which to do it, the proposition is comparatively simple, but if he has \$10 000 worth of work with only \$1 000 with which to do it, a very high grade of engineering ability is called for.

Engineering is a union of common sense and science. It deals with the forces of Nature; but by far the most difficult force with which the engineer has to deal is human nature, and ability to deal with human nature can hardly be acquired in any college course. It requires genius, to begin with, and proper training, which includes a good knowledge of the fundamental principles, including moral principles, and the ability to apply them effectively.

Mr. Durham's remarks bring out strongly the fact that although the organization in Paris is almost ideal, the results are open to criticism; whereas, in London, notwithstanding the fact that the organization contains obvious defects, the results are excellent; and in Berlin where everything is under the control of a Government which has a genius for organization, the results are variable. These facts point strongly to the indication that administration is a personal matter, rather than a question of form of organization, and, to produce the desired result, it must be sufficiently elastic to permit of all men being assigned to the duties for which they are best adapted.

The French are very systematic and competent, technically, but inferior to the English in executive ability. The Germans are very skillful and industrious, but are so accustomed to military rule that difficulty is apt to develop in case of an emergency requiring special methods.

The writer is often inclined to deplore the position of many in the Profession who advocate the placing of engineers at the head of all work that is even partly of an engineering nature. The combination of a high grade of technical skill and knowledge with the sure and accurate judgment of men that is required in any high administrative position, together with the proper understanding of the many legal, commercial, and political questions involved in State highway administration, is so rare as hardly to be looked for on this earth; and, for a man at the head of such an organization to be notably deficient in any of the latter requirements is almost certain to wreck the department; whereas, a man possessed of these qualifications, as well as the self-control and tact required for such a position, can usually make a success of it, even though he is equipped with but a moderate amount of technical knowledge. A man possessed of the latter, with a sufficiently elastic

organization, will have the power and the disposition to avail himself of any special knowledge or skill possessed by any member of his staff, whatever his title may be, in the solution of any problem that may present itself. Every man has his weaknesses and deficiencies, and it is of the utmost importance that the "Chief" see that none of them is allowed to interfere with the smooth running of the organization. Mr. Whited.

The writer would suggest that the most important qualifications of the head of any department of public works are tact, common sense, self-control, and the ability to deal with men, both personally and commercially, and a knowledge of his legal rights and duties, as well as those of the traveling public, the public service corporations, and the abutting property owners. The more knowledge of engineering technology he has the better, but the questions which arise in strictly administrative work are generally questions which must be settled on the spot, while those of a technical nature can usually be referred to some expert in paving, bridge construction, tests of materials, etc., who should be included in the staff of the department.

A word as to the politician: The general idea is that the politician is a professional criminal, whereas, the fact is that politicians of the higher grade are fully as honest as men in most other lines of business, and are not only possessed of tact and common sense, with a thorough knowledge of the world, but are also industrious as a rule.

As to the statement by Mr. Bennett, to the effect that a prominent engineer has said that there were few engineering problems to be faced in highway work, the writer would say that a good definition of an engineer is, a man who can do with one dollar, what any one can do with two; but, considering the many millions of dollars spent annually on road construction and maintenance, and the importance of the interests involved, there is ample scope for the exercise of the best grade of engineering ability.

It is doubtful whether anybody has reached, or ever will reach, a final settlement of the conflict between the inspector and the contractor's foreman, at least in the present state of human nature. The strictly competent inspector does not exist, and never will, and the same may be said of every one connected with the work. The only thing to do is to fight out each individual question as it arises. The writer will suggest, however, that a frequent cause of the differences between the contractor's foreman and the inspector of small and limited experience, is that the foreman, or some of his workmen, with or without the knowledge of his superiors, will in some way get the better of the inspector when he is off his guard, and the inspector finds it out when it is too late. The anger, suspicion, and hostility which takes possession of the inspector need not be explained to any one understanding human nature. It is easy to deceive anybody, but it is almost certain that it will be found out eventually.

Mr. Whited. Regarding efficiency boards, the writer would say that the efficiency expert has his place in all public or private work, but his outgivings can seldom be taken as final, because so much depends on the personality of the officials and employees, both high and low.

Mr. Connell. William H. Connell, Assoc. M. Am. Soc. C. E. (by letter).—The discussions on "Municipal Highway Engineering Organizations" would seem to indicate that there is a general tendency to raise the standard of organizations controlling this important branch of engineering work. The exceptions taken to the organization advocated by the writer are matters of minor detail, and in some cases are due to a misunderstanding. A few of those who discuss the subject are apparently under the impression that the writer advocated a standard highway organization. This, of course, is not the case, as such an organization would not be possible on account of the varying sizes of the municipalities; but an attempt was made to outline the general principles which could be followed in forming an organization suitable for municipalities of various sizes. Some seem to dwell at length on coping with existing conditions. That, of course, is necessary, but is simply routine; it would not do, under any conditions, to plan an ideal organization which could not be operated successfully under different laws, ordinances, and existing conditions, over which no control could be exercised; and, therefore, the first thing to do is to plan the work in such a way that it can be carried on successfully and economically under existing conditions, and try to have these conditions improved so that the ideal organization can be put into operation in the course of time.

One of the questions of detail, on which there seems to be some difference of opinion, is whether there should be a separate organization for maintenance. It would seem to be logical that the organization to take charge of the maintenance should be the one under which the pavements were constructed, as the men in that department understand all the conditions in connection with that work—some of which might at a later date be factors in the maintenance problem. Having a single organization also does away with the shifting of responsibility. The maintenance is just as important a part of the road problem as the construction, and no pavement should be laid without taking into consideration, among other things, its probable life and the probable maintenance charges during its life.

Another very important consideration, in having one organization in charge of both branches of the work, is that it tends to build up a better personnel and makes the men more generally useful. In other words, an important part of the education of the highway engineer is to understand and realize the importance of the maintenance problem. A good many of them do not realize the importance of this problem

because, in many cases, separate divisions are in charge of the construction and maintenance, and the men who have nothing to do with maintenance, as a rule, devote very little time and study to the subject. There is no doubt that more study would be given to the general highway problem if all the work was under one organization; the engineers who constructed the pavements would know that they were to be held responsible for the maintenance, and would naturally give more time and thought to this phase of the work. Aside from this fact, it would require a smaller organization if combined than if separate, as each division—no matter how well organized—tends to a duplication of work, to a certain degree. Mr. Connell.

One member advocates the general scheme of organization, but points out that the weak point of the system is in the question of personnel, and further states that in the United States there is no trained class of public works officials or Government engineers, organized like the French corps for bridges and highways. In view of this, he states that, in the light of possible conditions, it would seem to be better policy to concentrate along all classes of work, organizing the bureau in sections for doing specific things, in charge of the man best qualified to do them. This, of course, is a proposed scheme to cope with existing conditions, which is just what should be avoided. Engineers are often too apt to accept conditions as they are, instead of trying to improve them, and if they do not strive to form an organization that will stand and that is not dependent on any one man suited to fill a particular position, the organization can never be a stable one. A highway organization should not be dependent on any individual, but should be designed so that it would be constantly training men to fill the vacancies of those passing out, and it would appear to be better policy to endeavor to change the conditions that prevent formulating an organization of this kind, than to become reconciled to existing conditions.

A very important consideration in connection with the highway organization is the question of the *esprit de corps*. Mr. Meeker states that the key-note of all successful organizations is that all must work harmoniously. No matter how well the corps is organized, without harmony it will be a failure. This is a very important matter, and cannot be given too much attention as a factor in the control of a good working organization for this or work of any other class.

There was some discussion regarding the method of handling the patrol inspection. The writer advocated that the street-cleaning force should be combined with the general highway organization, and that the maintenance patrol inspection of the pavements should be handled by the street-cleaning inspectors or the foremen. The advisability of this method was questioned. This was advocated in order to increase

Mr. Connell. the usefulness of the men in charge of the inspection work, and thus reduce the cost of the patrol inspection by utilizing men already on the work, such as the street-cleaning inspection force, instead of employing additional men to cover the ground every day. There is no reason why the police could not also be utilized for work of this kind, and when our municipalities are further advanced, and more on a business basis, there is no doubt that they will be more generally used in connection with matters of this kind than at the present day.

If some general principles were formulated relative, not only to the character of the organization, but the details of conducting the work, resulting from discussions of this kind, they would carry a certain weight and would be a great help to engineers throughout the country in having the laws, regulations, and ordinances changed to make it possible to plan an organization in accordance with what is conceded to be the best practice by the authorities on this subject. In other words, it is always easier to put into effect anything that is the combined opinion of the engineers of the country at large than it is to accomplish it by individual effort.

FACTORS LIMITING THE SELECTION OF MATERIALS AND OF METHODS IN HIGHWAY CONSTRUCTION.

Mr. Washington. WILLIAM DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—Based on experience as a contractor and engineer, the speaker wishes to emphasize the importance which he attaches to traffic censuses in connection with this topic. In this matter, much may be learned from European engineers, as traffic censuses have been taken for many years in both France and England. Such censuses, and soil and geological surveys, should form an important part of the work preliminary to the design of any highway.

During its investigations in New York State, Commissioner Carlisle's Board of Consulting Engineers,* realizing the variety of local conditions obtaining in different parts of the State, recommended a "material" survey of the State, in order to make available to the Highway Department information of inestimable value if local materials were to be used economically.

As an example of proper utilization of local material might be cited the case of a section of Route 4, extending across the southern tier of counties of New York. It was found that an appropriation of $1 000 000 was available for the construction of more than 150 miles of road. In order to meet the demands of the people for a maximum amount of mileage to be constructed under this appropriation, the engineers had designed a single-course semi-Telford road. The local material available was a soft sandstone which would rapidly disin-

* Report of the Board of Consulting Engineers, 1913.

tegrate under wear and weather. Due to long hauls from railroad stations, it was found that the importation of limestone for the base and trap rock for the top would require the expenditure of a large sum of money. The Board of Consulting Engineers, therefore, decided to construct a cement concrete top, using for the mineral aggregate the local sandstone, on a base of sandstone, and to complete the pavement with a bituminous carpet. Mr. Washington.

Highway engineers require more definite knowledge relative to the life of various types of roads when subjected to varying local conditions. Massachusetts, apparently, is about the only State which has made a careful series of records covering the wearing qualities of certain classes of materials and types of roads under given traffic conditions. Every State should emulate Massachusetts and endeavor to ascertain, under their local conditions, the value of the materials which are available for road building and the service which a given type of road will give under known traffic, and careful records should be taken covering the rate of wear under traffic and the deterioration due to various climatic conditions.

Mr. Crosby has mentioned the subject of harmonic action and harmonic waves in road surfaces. This expression was evolved by that able engineer and road builder, Col. R. E. Crompton, of London, who has ascertained that roads have a tendency to creep or to form miniature hills and hollows. With certain types of traffic, he has found that the distances between the hills and hollows have a certain relative length. The speaker, in company with Col. Crompton, examined one road on which there were more than 1 000 motor busses per day, and found a series of long waves, which occurred with great regularity throughout the length of the road. The speaker believes that improper rolling of the road and the compression of the material are the main factors influencing the formation of harmonic waves. Traffic further develops the harmonic waves due primarily to the vibration of the motor cars passing over them. During the process of rolling it will be noticed that the material rises in front of the front roller and pushes ahead to a certain extent, finally ceasing to move forward, at which time the roller goes over it. If examined, the road surface will be found to have a marked swell. Col. Crompton has endeavored to solve this problem by devising a machine with three sets of rollers, instead of the ordinary tandem roller. The first and third rollers are somewhat lighter than the second one. The first roller consequently smooths and compresses the material, but does not put excessive weight on it. The second and third rollers give the additional compression necessary. It has been found possible to secure a much smoother base and surface for the road with this triple roller than with the ordinary tandem roller.

Mr. Green. P. E. Green, M. Am. Soc. C. E. (by letter).—It is gratifying to find such a general agreement as to the main points among the eminent engineers who have engaged in this discussion. Naturally, there have been some sharp disagreements, but these have been rather few. Apparently, road construction and maintenance are becoming standardized, a result to be desired in many ways.

Mr. Whinery seems to object mildly to the suggestion that the esthetic features of the highway should be considered, but it is believed that his objections are not well taken. It is so easy to consider them, the result is so desirable—often at little or no additional expense—that it would appear poor policy, both from a utilitarian and artistic point of view, to ignore such benefits. Too often engineers are justly charged with lack of appreciation of that very point.

It is believed also that Mr. Whinery is incorrect in his statement that the temporary and inadequate railways built in the early days resulted from the undeveloped conditions of the principles and requirements involved, rather than from financial or economic considerations. Undoubtedly, much poor engineering was perpetuated, but also much of it was of the highest order, in that it adapted its construction to its pocketbook. It is comparatively simple to build well with an unlimited pocketbook, but certainly a higher order of intelligence is often required to do well with little money.

As to the roadway diagram, Fig. 1, it would appear as if Mr. Whinery failed to use it with the discretion advocated by the writer. According to the diagram, the roadway width of the boulevard having 600 vehicles per foot of width should have been at least 80 ft., instead of 38 ft., as it actually is, and, if future traffic increase is taken into account (as it should be), the width would be increased accordingly. It cannot be maintained that the roadway width is a straight-line function of the traffic. It is believed Mr. Whinery would think better of the diagram in question if he studied it and its text a little more carefully.

Mr. Tillson encountered the same obstacle that the writer found; that is, the difficulty in describing "new matter", but he got around it more gracefully. However, the writer disclaims its authorship; it was simply one of the Society's rules governing this discussion.

Mr. Tillson further calls attention to the late practice of the City of Paris in treating paving blocks with oil under pressure to the amount of 10 or 12 lb. per cu. ft. This is largely for the purpose of preventing the blocks from swelling. The writer, however, believes that swelling will not be prevented in that manner. At Longview, Tex., in September, 1913, after a hot dry summer, it was found that blocks treated with true creosote oil in 1911 absorbed nearly 50% of their volume of water and had lost nearly 50% of the contained oil.*

**Engineering News*, December 4th, 1913.

It is believed that there is only one way to treat wood blocks permanently so that they will not absorb a great deal of water, and that is by traffic. Mr. Green.

The writer thoroughly agrees with Mr. Meeker and Mr. Washington that the so-called "harmonic" waves are caused more by poor workmanship and materials than anything else. On Diversey Boulevard, Chicago, paved with bituminous concrete in 1911, at a section having a mixed traffic of about 5 000 vehicles per day, evenly divided as to direction, one side has pushed into these waves so that it has become exceedingly rough. The other side is quite smooth. This condition is apparent for only about 2 000 ft. The rest of the highway (several miles) is in perfect condition. It is very evident that defective material or workmanship or both has been the cause of this.

EQUIPMENT AND METHODS FOR MAINTAINING BITUMINOUS SURFACES AND BITUMINOUS PAVEMENTS.

William de H. Washington, Assoc. M. Am. Soc. C. E.—Three members of this Society were engaged by the Highway Department of the State of New York to devise methods of maintaining highways and reducing the cost of maintenance. They concluded that the railroads had set a very excellent example as to the organization for upkeep, by which they put this work in the hands of a body of men under a competent foreman, instead of distributing their men individually over a section. These men work as a unit. Each group of men is provided with all necessary materials, which materials are usually carried on hand cars. Mr. Washington.

The speaker has devised a machine to accomplish exactly the same object in connection with the maintenance of roads, and meets the requirements mentioned in Mr. Farrington's paper. This movable repair plant consists of a motor truck equipped with all tools and machines necessary for the maintenance of highways, and supplied with storage capacity for broken stone, sand, and tar. The machines include a pressure spraying device, sand and stone chip distributor, a roller (beneath the body of the truck), a stone and sand dryer, a mixing machine, a road sweeper, a revolving power scarifier, extension arms for pulling a plow, scraper, split-log drag, or a snow plow, a power loader and derrick, and all requisite small tools. The truck has storage capacity for about 4 tons of broken stone and from 12 to 14 bbl. of tar. When used for the repair of macadam roads, a large removable water tank is carried on the truck.

The outfit has been designed with the intention of utilizing it in the maintenance of all types of roads and pavements constructed by the State Highway Department of New York.

Mr. Masury.

A. F. MASURY, ESQ. (by letter).—There is no equipment better adapted for the maintenance of bituminous surfaces and bituminous pavements, or for making roads, than motor trucks; and probably none is more generally in use for this purpose.

This discussion will give facts, based on the actual operation and maintenance of trucks, which will enable the engineer and contractor to operate them to the best advantage in road work.

Factors of Motor Truck Operation.—Glaring generalities have been characteristic of most discussions on motor truck operation. The motor truck salesman, with his optimistic figures and his aggressiveness to make a sale, has caused the engineer, contractor, and merchant to use his figures in planning truck operation. This, done without much thought on the part of the owner, really gives him the salesman's figures of past performances on which to base his intended operation.

The commercial motor vehicles are machines, made in a variety of types and sizes, designed for specific purposes, and must be built, their field of operation laid out, their operation controlled, and the upkeep looked after, by engineers.

The purpose of this paper is to present data on different sizes and types of motor trucks, to show actual figures as to their performance, and state the factors which must be used in operating them.

Arrangement of Space for Most Economical Storage.—In storing motor trucks, it has been worked out in practice that they can be handled best in a building of usual size by arranging them in two rows with an open aisle between; that is, the trucks are backed into place, with their fronts facing the aisle. This permits them to be taken out without any delay, and also allows them to be run into the aisle, where washing and drainage facilities should be provided.

In most buildings, posts are necessary, and should be toward the front of the trucks, but far enough back to allow the trucks to turn out into the aisle. The depth from the wall to the post should be at least 16 ft. 6 in.; thus, the front wheels of trucks of most sizes will project beyond the posts and allow an easy and quick turning into the aisle. The aisle should be about 27 ft. wide in order to provide a practical turning radius for getting out or backing in. The posts should be about 22 ft. apart, so as to allow the storage of three large trucks between them. Where a large number are stored the proportioning of small to large trucks allows this arrangement to be worked out with the economical use of all space.

The size of the doors should be looked into very carefully, as trucks, especially of the van type, are now very large. In order to take those of the largest type, such as vans and those used for crackers

and paper boxes, the doors should have a clearance of at least 12 ft. 3 in., and should be double, opening outward. The clearance between the butts of the doors should be at least 10 ft. 9 in. This allows sufficient space for the fastenings of the doors, without interfering with the trucks as they move in and out. There should be lights on the butts of the doors on each side, quite high up. Mr. Masury.

The elevators must also be considered carefully. They must be at least 24 ft. long and 11 ft. wide, in order to take the modern motor truck, and should have a carrying capacity of at least 7 tons, in order to accommodate the trucks when light. A heavier elevator, to take care of loaded trucks, is impractical, from a commercial standpoint; loaded trucks are not usually garaged in such buildings, but might be accommodated on the ground floor. The elevator should have four guiding rails, and a speed of about 60 ft. per min.

Service.—There must be adequate facilities for filling the tanks with gasoline, and oil, and for the storage of the large quantities of lubricants used in the operation of such trucks.

To comply with fire prevention regulations, the gasoline must be stored underground, and must be piped to at least two outlets on each garage floor. There are two practical ways of raising it to each floor: by hydraulic power and by pressure. Either of these systems can also be used for cylinder oil and kerosene. There should be at least one outlet for kerosene and one for oil on each floor. Meters should be attached to all outlets, and provision should be made for locking them. At least one portable tank should be provided for each floor, as trucks must be filled during the hours when the garage is most crowded, and when, at best, only two can be filled from each outlet.

Oil and kerosene may be carried to the trucks from the outlets in measures, as comparatively small quantities are used.

Transmission oil, cup grease, wood alcohol (for use in the radiators in cold weather), and other material and accessories may be stored in a room on the floor with the trucks.

There should be an efficient system for keeping account of the exact quantities of supplies consumed by each machine.

A dirty truck (and trucks collect dirt with astonishing rapidity) is much harder to keep in good order than a clean one, as dirt not only gets between moving parts and into bearings and causes wear, but it prevents the discovery of minor troubles until they have become major ones, and makes repairs a difficult and distasteful job if they are discovered. Therefore, trucks should be washed at night. For this purpose a regular force should be employed, its size depending on the number of cars to be washed. Four washers and two polishers can take care of fifty cars.

Mr. Masury.

Drivers are not always to be depended on to make repairs; their hours are long, and they have little time for them during the day's run. For this reason, if six or more trucks are being operated, it will be well to have a night repair man on duty, with one or two helpers. As the trucks come in, each driver should report any trouble with his machine. The repair man can also examine the trucks to discover loose bolts, and can grease them up.

Truck Sizes.—Trucks may be divided into three classes, according to the arrangement of the motor and the driver's seat:

Type 1.—This is the usual touring car type; the motor is over the front axle, the driver and control are behind the motor.

Type 2.—In this type the motor is over the axle; the driver and control are directly above the motor.

Type 3.—The motor is over the front axle; the driver and control are on one side of the motor, and the helper is on the other side.

Types 2 and 3 conserve the length of the chassis, as less space is taken for the driver and control, thus allowing a longer loading platform in proportion to the wheel base than Type 1.

The most important average dimensions for each size of truck are given in Table 2; and the diameters of turning circles are given in Table 4.

TABLE 2.—Most Important Average Dimensions of Trucks.

Type.	Capacity.	Wheel base.	Extreme length.	Length, back of cab.	Extreme width, in inches.	Height of Loading Platform, in Inches.		Clearance, in inches.
						Empty.	Loaded.	
Type 1	1-ton.	12 ft.	16 ft.	9 ft. 2 in.	68	36	32	9½
	2-ton.	13 ft.	17 ft.	9 ft. 2 in.	68	38	34	9½
	3-ton.	13 ft. 6 in.	20 ft. 6 in.	11 ft. 0 in. to 11 ft. 6 in.	87	42	38	10
	4-ton.	14 ft. 6 in.	21 ft. 6 in.	12 ft. 0 in. to 12 ft. 6 in.	87	42	38	10
	5-ton.	15 ft. 6 in.	22 ft. 6 in.	13 ft. 0 in. to 13 ft. 6 in.	87	44	38	10
	7½-ton.	16 ft 6 in.	23 ft. 6 in.	14 ft. 0 in. to 14 ft. 6 in.	87	44	38	10
Type 2	3-ton.	11 ft. 6 in.	18 ft. 3 in.	12 ft. 6 in.	87	42	38	10
	4-ton.	12 ft. 6 in.	30 ft. 3 in.	14 ft. 6 in.	87	42	38	10
	5-ton.	13 ft. 6 in.	22 ft. 3 in.	16 ft. 6 in.	87	44	38	10
	7½-ton.	14 ft.	23 ft. 3 in.	17 ft. 6 in.	87	44	38	10
Type 3	5-ton.	11 ft. 10½ in.	18 ft. 1 in.	14 ft.	93½	46	40	7½
	7-ton.	13 ft. 4½ in.	20 ft. 1 in.	16 ft.	93½	46	40	7½
	10-ton	14 ft. 2½ in.	22 ft. 1 in.	18 ft.	93½	46	40	7½
Power-dump truck, Type 1.	5-ton.	11 ft. 9 in.	16 ft. 5 in.	8 ft. 3 in.	87	44	39	10½
	7½-ton.	12 ft. 10 in.	17 ft. 5 in.	9 ft. 3 in.	87	44	39	10½

Factors in Cost of Operation of Motor Trucks.—Table 3 shows the cost of operation of trucks of standard capacities running at different mileages per day, and the factors in detail of each item of upkeep. These figures are taken from actual costs of operating trucks, and are nominal in every way. Interest is reckoned at 5% on half the cost of the truck; insurance at 2% on 80% of the cost; and amortization at 10 per cent. Mr. Masury.

Column letter in Table 3.		Formulas.
A	Capacity.	
B	Average cost of chassis, based on list prices of the number of manufacturers indicated by the figures in parentheses.	
D	Miles per hour.	
F	Driver's salary, per year.	
G	Cost of garaging, per month.	
H	Miles per day.	
I	Maintenance (small repairs, etc.), per year.	
J	Gasoline, per year.	
K	Oil and grease, per year.	
L	Tires, per year.	
M	Interest, per life.	$M = R\frac{B}{2} \times 5$ per cent.
N	Insurance, per life.	$N = R \times$ (80% of B) $\times 2$ per cent.
O	Depreciation, per life.	$O = \frac{B}{10\% \text{ of } R}$.
P	Cost, per year.	$P = F + 12\,G + I + J + K + L + \frac{M}{R} + \frac{N}{R} + \left(O = \frac{B}{10}\right)$.
Q	Miles of life at each mileage per day truck is assumed to run.	
R	Years of life.	$R = \frac{Q}{W}$.
S	Cost, per life.	$S = P \times R$.
T	Cost, per day.	$T = \frac{P}{300}$.
U	Cost of tires, per mile.	$U = \frac{L}{W}$.
V	Cost of gasoline, per mile.	$V = \frac{J}{W}$.
W	Miles per year.	$W = H \times 300$.

Mr. Masury.

TABLE 3.—FACTORS IN THE COST OF OPERATION OF MOTOR TRUCKS.

A	B	D	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W
Capacity.	Average cost of chassis.	Miles per hour.	Driver's salary, per year.	Garage, per month.	Miles per day.	Maintenance, per year.	Gasoline, per year.	Oil and grease, per year.	Tires, per year.	Interest, per life.	Insurance, per life.	Depreciation during life.	Cost, per year.	Miles of life at each mileage per year.	Life of truck, in years.	Cost, per life.	Cost, per day.	Tire cost per mile.	Gasoline, cost per mile.	Miles of use, per year.
1-ton......	$1 963	19	$780	20	40	$120	$300	$120	$360	$408.75	$260.70	$1 626	$2 197.78	100 000	8.3	$18 241.74	$7.33	$0.03	$0.025	12 000
	(12)		830		60	240	450	190	540	231.75	147.00	833	3 106.78	85 000	4.7	14 601.86	10.36	0.03	0.025	18 000
			900		90	360	675	270	810	117.76	75.00	426	3 531.78	65 000	2.4	8 476.27	11.77	0.03	0.025	27 000
1½-ton....	2 225	18	780	21	35	135	276	102	341	533.97	331.00	2 140	2 199.73	100 000	9.6	21 117.41	7.33	0.0325	0.0262	10 500
	(12)		830		50	270	395	150	488	317.06	202.00	1 270	2 698.73	85 000	5.7	15 922.51	9.00	0.0325	0.0263	15 000
			900		90	405	710	270	878	133.50	85.00	535	3 728.73	65 000	2.4	8 948.95	12.43	0.0325	0.0262	27 000
2-ton......	2 918	17	780	22	35	150	288	75	393	663.33	461.00	2 660	2 361.44	95 000	9.1	21 489.10	7.87	0.0375	0.0274	10 500
	(8)		830		50	300	415	150	563	408.52	283.00	1 818	3 067.44	85 000	5.6	17 177.66	10.22	0.0375	0.0276	15 000
			900		80	450	660	225	900	246.96	136.00	789	4 026.44	65 000	2.7	10 877.39	13.42	0.0375	0.0275	24 000
3-ton......	3 575	15	780	23	30	180	292	75	404	893.75	572.00	3 575	2 233.39	90 000	10.0	22 333.90	7.44	0.045	0.0324	9 000
	(6)		830		50	360	487	150	678	473.68	303.00	1 989	3 612.39	80 000	5.3	19 145.67	12.04	0.045	0.0324	15 000
			900		80	540	781	225	1 081	241.31	154.00	1 018	4 329.39	65 000	2.7	11 716.35	14.43	0.045	0.0324	24 000
4-ton......	4 066	14	780	24	30	240	360	75	450	1 016.50	650.00	4 066	2 761.31	90 000	10.0	27 613.10	9.20	0.05	0.04	9 000
	(3)		830		50	480	600	150	750	538.94	344.00	2 140	3 666.31	80 000	5.3	19 431.44	12.22	0.05	0.04	15 000
			900		70	720	840	225	1 025	364.95	195.00	1 210	4 566.31	65 000	3.0	13 698.93	15.22	0.05	0.04	21 000
5-ton......	4 500	12	800	25	30	240	406	90	540	1 125.00	720.00	4 500	2 920.50	90 000	10.0	29 205.00	9.74	0.06	0.045	9 000
	(10)		900		50	480	678	180	900	596.25	311.00	2 380	4 072.50	80 000	5.3	21 184.25	13.58	0.06	0.045	15 000
			1 000		70	720	948	262	1 260	337.50	216.00	1 350	5 124.50	65 000	3.0	15 373.50	17.08	0.06	0.045	21 000
7½-ton....	5 400	9	800	26	25	255	395	94	750	1 525.50	976.00	5 400	3 367.40	85 000	11.3	38 051.62	11.22	0.10	0.0526	7 500
	(2)		900		40	510	630	150	1 200	837.00	535.00	2 348	4 463.40	75 000	6.2	27 673.08	14.88	0.10	0.0525	12 000
10-ton......	5 500	7	800	28	25	270	468	94	1 200	1 553.75	994.00	5 500	3 943.50	85 000	11.3	44 561.55	13.15	0.16	0.0624	7 500
	(1)		900		40	540	750	150	1 920	852.50	545.00	3 410	5 371.50	75 000	6.2	33 303.30	17.90	0.16	0.0625	12 000

Mr. Masury.

TABLE 4.—Turning Circles.

Type.	Capacity.	Wheel base.	Diameter of turning circle, in feet.	Type.	Capacity.	Wheel base.	Diameter of turning circle, in feet.
Type 1...	1-ton....	10 ft. 6 in.	47	Type 1...	5-ton....	12 ft. 6 in.	57
		11 ft. 6 in.	53			13 ft. 6 in.	62
		12 ft. 6 in.	55			14 ft. 6 in.	68
Type 1...	1½-ton..	10 ft. 6 in.	47	Type 1...	7½-ton..	12 ft. 6 in.	57
		11 ft. 6 in.	53			13 ft. 6 in.	62
		12 ft. 6 in.	55			14 ft. 6 in.	68
		13 ft. 6 in.	60			15 ft. 6 in.	71
Type 1...	2-ton....	10 ft. 6 in.	47	Type 2...	3-ton....	11 ft.	47
		11 ft. 6 in.	53			11 ft. 6 in.	49
		12 ft. 6 in.	55			12 ft.	51
		13 ft. 6 in.	60	Type 2...	5-ton......	11 ft. 6 in.	53
Type 1...	3-ton....	12 ft. 6 in.	55				
		13 ft. 6 in.	59	Tractor..	3-ton......	10 ft. 4 in.	40
		14 ft. 6 in.	61	Tractor..	5-ton......	10 ft. 8 in.	41
Type 1...	4-ton....	12 ft. 6 in.	55	The trailer will usually cut under.			
		13 ft. 6 in.	59				
		14 ft. 5 in.	61				

TABLE 5.—Tire Equipment for Trucks.

(1) Truck Capacity.	(2) Axle load.	(3) Size of tires.	(4) $\frac{L}{W \times C}$	(5) Speed, in miles per hour.	(6) Mileage.	(7) Consumer's price.
1-ton..........	Front 2 600	36-4 single	18	16.5	7 000	$ 54.15
	Rear 5 200	36-5 single	28.8			70.45
	Rear 5 200	36-2½ dual	28.8			51.30
1½-ton........	Front 2 800	36-4 single	19.4	15	7 000	54.15
	Rear 6 300	36-3 dual	29.2			68.80
2-ton..........	Front 3 060	36-4 single	20.8	14	7 000	54.15
	Rear 7 200	36-3½ dual	28.6			83.67
3-ton, Type 1........	Front 4 300	36-5 single	23.9	12½	7 000	70.45
	Rear 10 800	36-4 dual	37.5			108.35
	Rear 10 800	42-4 dual	42.1			126.17
3-ton, Type 2........	Front 5 950	36-5 single	33.0	12½	7 000	70.45
	Rear 9 600	36-4 dual	33.3			108.35
	Rear 9 600	42-4 dual	28.5			126.17
4-ton..........	Front 4 950	36-6 single	22.9	11	7 000	87.97
	Rear 12 950	36-5 dual	36.0			138.90
	Rear 12 950	42-5 dual	30.8			172.40
5-ton, Type 1........	Front 5 300	36-5 single	29.4	10	7 000	70.45
	Rear 15 200	36-5 dual	42.2			138.90
	Rear 15 200	42-5 dual	36.2			178.40
5-ton, Type 2........	Front 7 425	36-6 single	34.4	10	7 000	87.97
	Rear 15 425	36-5 dual	42.8			138.90
	Rear 15 425	42-5 dual	36.7			172.40
7½-ton........	Front 6 175	36-6 single	28.6	7½	7 000	87.47
	Rear 20 325	36-6 dual	49.3			175.97
	Rear 20 325	42-6 dual	42.2			205.85

In Table 3 the cost of the body is left out intentionally, as it usually represents the amount of discount given by the manufacturer, and it needs but few repairs.

Tire Data.—The data on tire equipment in Table 5 show the average of the requirements and guaranties of the different tire man-

Mr. Masury. ufacturers, and also the average cost of tires to the consumer. Column 2, the axle load, gives the average weight on the front and rear axle of trucks of each size. Column 3, the size of tire, shows the sizes usually recommended by tire manufacturers for the respective loads carried on each axle. Column 4, $\frac{L}{W \times C}$, means the load on the axle, in pounds, divided by the width of the tire (the width being taken at the place where the tire meets the ground); C is the circumference of the wheel. The figures in this column are pounds.

This factor takes into account the size of the wheel, and therefore reconciles the fact that a larger wheel can have a tire of a smaller size, and is a better factor to use in comparing tire sizes than the weight, in pounds per square inch, of tire in contact with road.

Column 5 shows the normal speed of trucks of each capacity. Column 6 shows the guaranteed mileage by the tire manufacturers. Column 7 shows the average price at which tires can be bought by the consumer.

Mr. Farrington. W. R. Farrington, M. Am. Soc. C. E. (by letter).—The writer notes that considerable attention has been given by those taking part in the discussion on "Methods and Equipment for Maintaining Bituminous Surfaces and Bituminous Pavements" to the use of motor equipment.

Not much has been done, as yet, with trucks in State highway maintenance work in Massachusetts. Trucks have been used in some cases, but, except for transporting broken stone, gravel, bituminous materials, etc., over long hauls, apparently have not demonstrated their economy under existing conditions. The writer has no doubt, however, that, as this work increases and longer sections come under maintenance, motor trucks with the necessary equipment for patching, etc., will prove economical and will come into general use.

As to the material truck described by Mr. Washington, though the writer has no doubt that this will be useful, there would seem to be some objection to its general adoption at present, as it is probable that many of the culverts and bridges, even on main lines, are not strong enough to carry a truck of this weight, and there is also some question as to the effect on the road surfaces.

Recognizing the injurious effect of very heavy trucks on roads, where the surfaces and foundations, as well as culverts and bridges, are not especially designed for such traffic, the Massachusetts Legislature in 1913 passed a law prohibiting the use on the highways of trucks weighing more than 14 tons, without a permit from the State, city, or town officials having charge of repairs on the highways, and it would not seem to be good policy for the State to use on its work trucks of greater weight than the public generally is allowed to use.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

MEASUREMENT OF THE FLOW OF STREAMS BY APPROVED FORMS OF WEIRS WITH NEW FORMULAS AND DIAGRAMS.

Discussion.*

BY RICHARD R. LYMAN, ASSOC. M. AM. SOC. C. E.†

RICHARD R. LYMAN, ASSOC. M. AM. SOC. C. E. (by letter).—Further thought and study, and the matter presented in the discussions on this paper, have confirmed the writer's conviction that the weir without end contractions is the weir which, in actual practice, will measure water in the most satisfactory way. When it is used, the diagrams‡ and tables presented in this paper give results, without computation, in accurate accord with existing experimental data. They give results, too, for a great variety of weirs, covering a wide range of heads, without that "troublesome correction" for velocity of approach. Mr. Lyman.

Table Giving Discharges for all Weirs Without End Contractions, Within the Limits of Experimental Data.—For some purposes, tables giving discharges are more convenient than diagrams, therefore, discharges for a series of definite heads are presented in Table 69. Discharges for heads between those given may be found by interpolation.

Table Based on Formulas, Giving Discharges for High Heads.—A table of discharges, for heads higher than those for which accurate experimental data exist, by Gardner S. Williams, M. Am. Soc. C. E., has been published.§ The results are identical with those found by the use of Plate LXXXII. Therefore, no table for these discharges is presented in this paper.

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* Continued from April, 1914, *Proceedings*.

† Author's closure.

‡ Blueprints (on cloth) of the original diagrams, and of diagrams more closely ruled, are made for field use.

§ "American Civil Engineers' Pocket Book", pp. 867-868.

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TABLE 69.—DISCHARGES OVER SHARP-CRESTED WEIRS OF VARIOUS HEIGHTS WITHOUT END CONTRACTIONS.

These Figures were Taken from Plate LXXX, and are Therefore Based on the Experiments Made by Bazin, Francis, Fteley and Stearns, and Also on Those Made in the Hydraulic Laboratories of Cornell University and the University of Utah. The Limits Included in the Diagram and the Table are Fully Covered by the Original Experiments.

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
2 3/8	0.200	0.315	0.314	0.313	0.312	0.311	0.310	0.309	
2 7/16	0.205	0.327	0.326	0.325	0.324	0.323	0.322	0.321	
2 1/2	0.210	0.340	0.337	0.336	0.335	0.334	0.333	0.332	
2 9/16	0.215	0.352	0.351	0.350	0.348	0.347	0.346	0.346	
2 5/8	0.220	0.365	0.363	0.360	0.359	0.357	0.356	0.355	
2 11/16	0.225	0.377	0.375	0.372	0.370	0.369	0.368	0.367	
2 3/4	0.230	0.392	0.388	0.385	0.383	0.382	0.381	0.380	
2 13/16	0.235	0.404	0.400	0.398	0.396	0.394	0.393	0.392	
2 7/8	0.240	0.420	0.415	0.412	0.408	0.406	0.405	0.404	
2 15/16	0.245	0.433	0.427	0.425	0.422	0.420	0.417	0.416	
3	0.250	0.446	0.442	0.438	0.435	0.434	0.432	0.430	
3 1/16	0.255	0.460	0.453	0.450	0.447	0.445	0.443	0.442	
3 1/8	0.260	0.475	0.468	0.465	0.460	0.458	0.456	0.455	
3 3/16	0.265	0.490	0.483	0.478	0.475	0.473	0.470	0.468	
3 1/4	0.270	0.503	0.497	0.493	0.488	0.486	0.484	0.483	
3 5/16	0.275	0.515	0.508	0.505	0.501	0.498	0.496	0.495	
3 3/8	0.280	0.530	0.524	0.518	0.514	0.510	0.507	0.506	
3 7/16	0.285	0.546	0.537	0.532	0.526	0.523	0.520	0.517	
3 1/2	0.290	0.560	0.552	0.547	0.544	0.540	0.535	0.533	
3 9/16	0.295	0.576	0.566	0.560	0.555	0.552	0.548	0.546	
3 5/8	0.300	0.595	0.584	0.576	0.570	0.566	0.563	0.560	
3 5/8	0.305	0.610	0.595	0.588	0.582	0.577	0.575	0.572	
3 11/16	0.310	0.625	0.612	0.605	0.598	0.595	0.590	0.586	
3 3/4	0.315	0.640	0.627	0.620	0.613	0.608	0.605	0.602	
3 13/16	3.020	0.655	0.645	0.636	0.630	0.625	0.620	0.617	
3 7/8	0.325	0.670	0.655	0.650	0.641	0.636	0.632	0.630	
3 15/16	0.330	0.690	0.672	0.665	0.656	0.652	0.647	0.645	
4	0.335	0.705	0.690	0.680	0.670	0.665	0.660	0.657	
4 1/16	0.340	0.720	0.705	0.697	0.688	0.683	0.675	0.673	
4 1/8	0.345	0.738	0.720	0.710	0.703	0.696	0.692	0.687	
4 3/16	0.350	0.755	0.735	0.726	0.717	0.712	0.705	0.702	
4 1/4	0.355	0.770	0.752	0.743	0.732	0.725	0.720	0.717	
4 5/16	0.360	0.790	0.772	0.760	0.750	0.745	0.737	0.733	
4 3/8	0.365	0.805	0.786	0.775	0.764	0.757	0.750	0.746	
4 7/16	0.370	0.824	0.802	0.792	0.780	0.775	0.766	0.762	
4 1/2	0.375	0.840	0.817	0.805	0.795	0.790	0.782	0.777	
4 9/16	0.380	0.860	0.836	0.825	0.813	0.805	0.798	0.795	
4 5/8	0.385	0.875	0.853	0.840	0.826	0.820	0.810	0.806	
4 11/16	0.390	0.896	0.870	0.857	0.845	0.837	0.830	0.825	
4 3/4	0.395	0.910	0.885	0.870	0.860	0.852	0.845	0.838	
4 13/16	0.400	0.930	0.905	0.893	0.875	0.870	0.860	0.855	0.850
4 7/8	0.405	0.950	0.922	0.910	0.895	0.885	0.875	0.870	0.860
4 15/16	0.410	0.970	0.940	0.925	0.910	0.903	0.895	0.885	0.876
5	0.415	0.990	0.956	0.943	0.925	0.917	0.908	0.903	0.895
5 1/16	0.420	1.005	0.975	0.958	0.943	0.935	0.924	0.917	0.910
5 1/8	0.425	1.020	0.995	0.977	0.963	0.952	0.942	0.935	0.926
5 1/8	0.430	1.045	1.010	0.996	0.980	0.970	0.957	0.952	0.945
5 3/16	0.435	1.065	1.030	1.010	0.996	0.986	0.975	0.970	0.960
5 1/4	0.440	1.083	1.045	1.026	1.010	1.000	0.992	0.985	0.976
5 5/16	0.445	1.100	1.063	1.045	1.026	1.015	1.005	1.000	0.994
5 3/8	0.450	1.120	1.080	1.060	1.040	1.030	1.015	1.010	1.030
5 7/16	0.455	1.140	1.100	1.080	1.057	1.047	0.035	1.023	1.016
5 1/2	0.460	1.164	1.125	1.105	1.085	1.074	1.056	1.050	1.043

TABLE 69.—(*Continued.*)

Mr. Lyman.

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
5 9/16	0.465	1.185	1.140	1.120	1.100	1.090	1.075	1.067	1.057
5 5/8	0.470	1.205	1.163	1.143	1.120	1.106	1.095	1.085	1.077
5 11/16	0.475	1.230	1.185	1.162	1.140	1.125	1.110	1.105	1.096
5 3/4	0.480	1.250	1.205	1.185	1.160	1.150	1.133	1.125	1.115
5 13/16	0.485	1.270	1.223	1.200	1.175	1.163	1.150	1.140	1.130
5 7/8	0.490	1.290	1.245	1.220	1.200	1.183	1.166	1.160	1.150
5 5/16	0.495	1.310	1.265	1.233	1.215	1.200	1.186	1.176	1.166
6	0.500	1.335	1.285	1.263	1.235	1.220	1.203	1.195	1.185
6 1/16	0.505	1.355	1.300	1.280	1.250	1.236	1.220	1.210	1.202
6 1/8	0.510	1.370	1.320	1.296	1.270	1.257	1.237	1.225	1.220
6 3/16	0.515	1.390	1.340	1.317	1.287	1.274	1.255	1.244	1.235
6 1/4	0.520	1.415	1.360	1.335	1.305	1.290	1.273	1.260	1.252
6 5/16	0.525	1.440	1.380	1.355	1.325	1.310	1.290	1.280	1.274
6 3/8	0.530	1.465	1.405	1.375	1.346	1.330	1.310	1.300	1.293
6 7/16	0.535	1.490	1.425	1.400	1.365	1.353	1.335	1.320	1.310
6 1/2	0.540	1.510	1.440	1.415	1.385	1.365	1.350	1.336	1.327
6 9/16	0.545	1.530	1.465	1.435	1.403	1.385	1.365	1.355	1.345
6 9/16	0.550	1.555	1.490	1.460	1.425	1.405	1.385	1.370	1.365
6 5/8	0.555	1.575	1.505	1.475	1.440	1.420	1.400	1.390	1.380
6 11/16	0.560	1.595	1.525	1.495	1.460	1.435	1.415	1.405	1.395
6 3/4	0.565	1.616	1.545	1.515	1.475	1.455	1.435	1.420	1.410
6 13/16	0.570	1.640	1.570	1.535	1.500	1.475	1.455	1.440	1.430
6 7/8	0.575	1.665	1.590	1.555	1.517	1.500	1.475	1.460	1.450
6 15/16	0.580	1.686	1.610	1.576	1.537	1.517	1.495	1.480	1.470
7	0.585	1.713	1.635	1.605	1.565	1.540	1.520	1.505	1.495
7 1/16	0.590	1.740	1.670	1.630	1.590	1.570	1.545	1.530	1.523
7 1/8	0.595	1.760	1.685	1.650	1.605	1.585	1.560	1.543	1.535
7 3/16	0.600	1.790	1.700	1.675	1.625	1.605	1.580	1.565	1.555
7 1/4	0.605	1.805	1.730	1.695	1.655	1.627	1.605	1.590	1.580
7 5/16	0.610	1.830	1.750	1.715	1.675	1.650	1.625	1.610	1.600
7 3/8	0.615	1.855	1.775	1.735	1.695	1.675	1.650	1.630	1.620
7 7/16	0.620	1.880	1.795	1.760	1.710	1.690	1.670	1.650	1.640
7 1/2	0.625	1.905	1.815	1.780	1.730	1.705	1.685	1.670	1.665
7 9/16	0.630	1.930	1.845	1.805	1.760	1.730	1.705	1.694	1.687
7 5/8	0.635	1.955	1.875	1.835	1.785	1.760	1.725	1.710	1.700
7 11/16	0.640	1.980	1.900	1.860	1.815	1.790	1.760	1.740	1.730
7 3/4	0.645	2.010	1.915	1.870	1.820	1.800	1.770	1.750	1.740
7 13/16	0.650	2.035	1.930	1.890	1.840	1.810	1.780	1.760	1.750
7 7/8	0.655	2.060	1.960	1.915	1.860	1.830	1.805	1.785	1.775
7 15/16	0.660	2.085	1.985	1.945	1.890	1.865	1.830	1.815	1.805
8	0.665	2.110	2.005	1.965	1.910	1.880	1.850	1.830	1.820
8 1/16	0.670	2.135	2.025	1.980	1.930	1.900	1.870	1.850	1.840
8 1/16	0.675	2.160	2.055	2.000	1.945	1.910	1.880	1.860	1.850
8 1/8	0.680	2.185	2.075	2.030	1.980	1.945	1.910	1.895	1.885
8 3/16	0.685	2.210	2.095	2.050	1.990	1.960	1.925	1.905	1.895
8 1/4	0.690	2.240	2.125	2.075	2.025	1.990	1.960	1.935	1.925
8 5/16	0.695	2.260	2.150	2.095	2.040	2.005	1.970	1.945	1.930
8 3/8	0.700	2.295	2.180	2.130	2.070	2.030	1.995	1.975	1.965
8 7/16	0.705	2.325	2.200	2.155	2.100	2.065	2.025	2.000	1.985
8 1/2	0.710	2.350	2.220	2.170	2.115	2.085	2.040	2.020	2.005
8 9/16	0.715	2.380	2.250	2.195	2.140	2.105	2.060	2.035	2.025
8 5/8	0.720	2.410	2.275	2.220	2.160	2.125	2.085	2.060	2.045
8 11/16	0.725	2.435	2.300	2.245	2.180	2.155	2.115	2.090	2.080
8 3/4	0.730	2.465	2.325	2.270	2.200	2.175	2.135	2.110	2.095
8 13/16	0.735	2.490	2.350	2.295	2.230	2.190	2.150	2.130	2.120
8 7/8	0.740	2.520	2.375	2.320	2.250	2.210	2.170	2.140	2.130
8 15/16	0.745	2.550	2.405	2.340	2.275	2.235	2.200	2.170	2.160
9	0.750	2.585	2.430	2.375	2.300	2.260	2.225	2.190	2.180
9 1/16	0.755	2.605	2.455	2.400	2.325	2.285	2.245	2.220	2.200
9 1/8	0.760	2.640	2.480	2.415	2.340	2.300	2.270	2.240	2.230
9 3/16	0.765	2.670	2.510	2.440	2.370	2.320	2.300	2.265	2.255
9 1/4	0.770	2.700	2.540	2.470	2.400	2.350	2.320	2.285	2.275
9 5/16	0.775	2.730	2.560	2.500	2.420	2.375	2.330	2.310	2.300
9 3/8	0.780	2.760	2.590	2.515	2.440	2.400	2.345	2.330	2.325
9 7/16	0.785	2.790	2.610	2.550	2.460	2.415	2.365	2.345	2.335
9 1/2	0.790	2.820	2.630	2.570	2.480	2.430	2.380	2.360	2.350
9 9/16	0.795	2.850	2.660	2.595	2.510	2.460	2.410	2.380	2.365

Mr. Lyman.

TABLE 69.—(*Continued.*)

Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
9 9/16	0.800	2.890	2.700	2.625	2.550	2.500	2.440	2.410	2.400
9 5/8	0.805	2.910	2.730	2.660	2.575	2.520	2.465	2.425	2.410
9 11/16	0.810	2.940	2.755	2.680	2.595	2.545	2.485	2.445	2.425
9 3/4	0.815	2.975	2.780	2.700	2.610	2.565	2.505	2.460	2.440
9 13/16	0.820	3.010	2.810	2.735	2.640	2.590	2.530	2.500	2.480
9 7/8	0.825	3.045	2.840	2.770	2.670	2.610	2.560	2.530	2.510
9 15/16	0.830	3.070	2.870	2.790	2.700	2.640	2.580	2.550	2.535
10	0.835	3.100	2.905	2.830	2.730	2.675	2.610	2.580	2.565
10 1/16	0.840	3.130	2.930	2.840	2.760	2.695	2.630	2.600	2.590
10 1/8	0.845	3.160	2.950	2.880	2.785	2.730	2.650	2.615	2.605
10 3/16	0.850	3.190	2.990	2.910	2.800	2.750	2.680	2.650	2.630
10 1/4	0.855	3.230	3.015	2.930	2.840	2.780	2.710	2.670	2.650
10 5/16	0.860	3.260	3.040	2.960	2.860	2.800	2.735	2.700	2.680
10 3/8	0.865	3.290	3.070	2.980	2.880	2.815	2.750	2.715	2.695
10 7/16	0.870	3.320	3.100	3.010	2.910	2.840	2.780	2.740	2.720
10 1/2	0.875	3.350	3.120	3.035	2.930	2.870	2.795	2.765	2.750
10 9/16	0.880	3.395	3.160	3.070	2.965	2.900	2.820	2.790	2.780
10 5/8	0.885	3.415	3.180	3.090	2.980	2.920	2.840	2.810	2.790
10 11/16	0.890	3.445	3.200	3.120	3.010	2.940	2.860	2.825	2.820
10 3/4	0.895	3.480	3.235	3.150	3.040	2.970	2.895	2.860	2.845
10 13/16	0.900	3.520	3.270	3.180	3.070	3.000	2.920	2.890	2.870
10 7/8	0.905	3.550	3.300	3.210	3.100	3.035	2.940	2.910	2.890
10 15/16	0.910	3.580	3.330	3.235	3.120	3.055	2.970	2.930	2.910
11	0.915	3.620	3.360	3.260	3.155	3.085	3.000	2.955	2.935
11 1/16	0.920	3.655	3.390	3.290	3.180	3.110	3.030	2.980	2.960
11 1/8	0.925	3.690	3.420	3.325	3.210	3.140	3.055	3.010	2.990
11 1/8	0.930	3.720	3.445	3.350	3.230	3.160	3.075	3.030	3.010
11 3/16	0.935	3.760	3.480	3.380	3.250	3.180	3.100	3.060	3.040
11 1/4	0.940	3.800	3.510	3.405	3.290	3.210	3.130	3.080	3.060
11 5/16	0.945	3.830	3.540	3.430	3.315	3.240	3.150	3.110	3.090
11 3/8	0.950	3.870	3.580	3.470	3.350	3.260	3.180	3.140	3.120
11 7/16	0.955	3.900	3.610	3.500	3.380	3.295	3.200	3.165	3.140
11 1/2	0.960	3.940	3.640	3.540	3.400	3.325	3.235	3.190	3.170
11 9/16	0.965	3.980	3.680	3.570	3.430	3.355	3.260	3.210	3.190
11 5/8	0.970	4.010	3.700	3.590	3.450	3.370	3.275	3.235	3.200
11 11/16	0.975	4.040	3.740	3.625	3.490	3.405	3.310	3.270	3.250
11 3/4	0.980	4.080	3.770	3.650	3.520	3.430	3.330	3.290	3.270
11 13/16	0.985	4.120	3.800	3.690	3.555	3.460	3.365	3.320	3.300
11 7/8	0.990	4 150	3.830	3.710	3.580	3.480	3.380	3.340	3.320
11 15/16	0.995	4.180	3.850	3.730	3.590	3.510	3.400	3.360	3.330
12	1.000	4.230	3.900	3.780	3.640	3.555	3.440	3.400	3.375
12 1/8	1.010	4.300	3.970	3.840	3.710	3.600	3.500	3.450	3.420
12 1/4	1.020	4.380	4.030	3.900	3.760	3.670	3.560	3.500	3.480
12 3/8	1.030	4.450	4.100	3.970	3.820	3.720	3.600	3.560	3.540
12 1/2	1.040	4.520	4.170	4.040	3.880	3.780	3.670	3.620	3.590
12 5/8	1.050	4.610	4.240	4.120	3.950	3.850	3.730	3.670	3.650
12 11/16	1.060	4.800	4.320	4.180	4.020	3.910	3.790	3.740	3.710
12 13/16	1.070	4.760	4.370	4.220	4.070	3.960	3.830	3.770	3.750
12 15/16	1.080	4.820	4.430	4.280	4.130	4.010	3.890	3.820	3.800
13 1/16	1.090	4.900	4.480	4.340	4.180	4.060	3.930	3.870	3.840
13 3/16	1.100	4.980	4.570	4.420	4.240	4.140	3.990	3.940	3.910
13 5/16	1.110	5.060	4.640	4.480	4.320	4.190	4.060	4.000	3.960
13 7/16	1.120	5.150	4.710	4.560	4.370	4.240	4.120	4.050	4.010
13 9/16	1.130	5.220	4.780	4.610	4.420	4.300	4.170	4.100	4.070
13 11/16	1.140	5.300	4.840	4.670	4.480	4.360	4.210	4.160	4.130
13 13/16	1.150	5.380	4.910	4.740	4.560	4.420	4.270	4.210	4.180
13 15/16	1.160	5.450	4.980	4.800	4.610	4.480	4.330	4.260	4.220
14 1/16	1.170	5.510	5.050	4.870	4.670	4.540	4.380	4.320	4.280
14 1/8	1.180	5.600	5.130	4.950	4.740	4.610	4.440	4.380	4.340
14 1/4	1.190	5.680	5.200	5.000	4.800	4.660	4.500	4.420	4.400
14 3/8	1.200	5.780	5.250	5.075	4.870	4.720	4.560	4.480	4.440
14 1/2	1.210	5.860	5.340	5.150	4.940	4.780	4.610	4.540	4.500
14 5/8	1.220	5.940	5.420	5.250	5.000	4.860	4.680	4.610	4.590
14 3/4	1.230	6.000	5.460	5.270	5.050	4.910	4.720	4.640	4.610
14 7/8	1.240	6.100	5.550	5.360	5.150	4.980	4.800	4.720	4.680
15	1.250	6.200	5.620	5.430	5.220	5.050	4.860	4.780	4.740
15 1/8	1.260	6.275	5.675	5.500	5.275	5.100	4.910	4.830	4.800

TABLE 69.—(*Continued.*)

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Head, in inches.	Head, in feet.	Weir 0.5 ft. high.	Weir 0.75 ft. high.	Weir 1.00 ft. high.	Weir 1.50 ft. high.	Weir 2.00 ft. high.	Weir 3.00 ft. high.	Weir 4.00 ft. high.	Weir 6.00 ft. high.
15 1/4	1.270		5.750	5.560	5.325	5.180	4.970	4.890	4.850
15 3/8	1.280		5.820	5.620	5.380	5.225	5.000	4.940	4.900
15 1/2	1.290		5.900	5.680	5.450	5.275	5.075	5.000	4.960
15 5/8	1.300		5.975	5.775	5.525	5.350	5.150	5.050	5.020
15 11/16	1.310		6.060	5.850	5.600	5.425	5.225	5.130	5.080
15 13/16	1.320		6.150	5.920	5.675	5.500	5.275	5.200	5.150
15 15/16	1.330		6.200	6.000	5.730	5.550	5.350	5.250	5.220
16 1/16	1.340		6.300	6.050	5.800	5.620	5.400	5.320	5.260
16 3/16	1.350		6.375	6.130	5.875	5.675	5.460	5.370	5.320
16 5/16	1.360		6.450	6.200	5.940	5.750	5.520	5.430	5.380
16 7/16	1.370		6.505	6.300	6.000	5.820	5.580	5.500	5.450
16 9/16	1.380		6.625	6.375	6.080	5.900	5.650	5.560	5.525
16 11/16	1.390		6.700	6.450	6.150	5.960	5.725	5.625	5.575
16 13/16	1.400		6.780	6.530	6.230	6.040	5.770	5.675	5.640
16 15/16	1.410		6.860	6.620	6.320	6.100	5.850	5.760	5.700
17 1/16	1.420		6.950	6.675	6.375	6.150	5.920	5.820	5.760
17 1/8	1.430		7.000	6.750	6.450	6.220	5.975	5.875	5.825
17 1/4	1.440		7.075	6.820	6.520	6.300	6.030	5.930	5.880
17 3/8	1.450		7.150	6.900	6.600	6.360	6.100	6.000	5.950
17 1/2	1.460		7.250	6.975	6.660	6.430	6.150	6.050	6.000
17 5/8	1.470		7.330	7.050	6.740	6.500	6.220	6.120	6.060
17 3/4	1.480		7.400	7.130	6.800	6.508	6.300	6.175	6.125
17 7/8	1.490		7.480	7.200	6.850	6.640	6.330	6.230	6.160
18	1.500		7.600	7.300	6.950	6.720	6.420	6.300	6.250
18 1/8	1.510		7.660	7.360	7.020	6.775	6.500	6.360	6.300
18 1/4	1.520		7.750	7.450	7.100	6.850	6.550	6.450	6.360
18 3/8	1.530		7.825	7.520	7.160	6.930	6.640	6.520	6.460
18 1/2	1.540		7.900	7.600	7.230	7.000	6.680	6.575	6.500
18 5/8	1.550		7.980	7.660	7.300	7.040	6.740	6.625	6.560
18 11/16	1.560		8.075	7.730	7.400	7.120	6.800	6.700	6.630
18 13/16	1.570		8.150	7.820	7.450	7.180	6.860	6.740	6.680
18 15/16	1.580		8.250	7.900	7.525	7.250	6.940	6.800	6.750
19 1/16	1.590		8.300	7.960	7.560	7.300	6.975	6.850	6.780

Table Giving Discharges for Weirs with Broad Crests.—Discharges over broad-crested weirs are given in Table 70. The matter it contains was prepared from Plate LXXXIII.

Suggestive Details for Weirs Recommended.—Suggestive details to be used in constructing weirs without end contractions are presented in Figs. 50, 51, and 52. It is recommended that the angle iron proposed for the crest have its end securely embedded in the walls of the channel; also that flush-gates, as shown in Figs. 50 and 51, be constructed so that the bottoms of the openings they cover will be on a level with the top of the floor of the channel of approach. If these flush-gates are of thin material, they can perhaps be located on the up-stream side of the bulkhead so that they will not affect materially the accuracy of the measurements. If supplied with lugs or iron loops, they may be pulled up and pushed down at pleasure.

Flush-Gates for Small Weirs.—For smaller weirs, the writer has used flush-gates of 1-in. lumber, with sheets of rubber gasket attached to them in such a way that they form a sort of flap or "check-valve". The pressure of the water on only one side of the gasket makes it fit close against the bottom of the channel and also around the three

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Quantity of Water Each Weir Will Measure, and Quantity of Concrete Required to Construct Each Weir.—Suggestive dimensions of weirs to be used for measuring definite quantities of water and also the number of yards of concrete required to build these structures are given in Table 71.

The horizontal notch shown just below the crest of the weir and extending down stream to the end of the wall is constructed to allow the air free access to the under side of the falling sheet. Conditions are often such that a hole may be made through the wall of the channel just down stream from and below the crest of the weir, as at *A*, Fig. 51. This feature must not be overlooked when accurate measurements are desired.

Comparison of Results Given by the Diagrams and those Computed by the Francis Formula.—Mr. Moritz presents two ingenious and very convenient diagrams for finding the discharge over two weirs by the Francis formula. The scale of his diagram for suppressed weirs, however, is small. In order, therefore, to compare its results with those given by the diagrams presented in this paper, the quantities used have been computed by the Francis formula. The results of this comparison are given in Table 72.

The figures above the heavy line are the results obtained under the conditions specified by Mr. Francis himself, *viz.*, that with a "depth of channel leading to the weir * * * as small as three times that on the weir" the Francis formula "agrees with experiment within less than one per cent.", therefore "if the canal leading to the weir has a suitable depth, it will be requisite only when great precision is required * * * to go through the troublesome calculation of correcting the depth on the weir".*

Table 72 shows that, even when the conditions specified by Mr. Francis himself are satisfied (conditions which all experienced engineers attempt to follow), differences occur, varying from 1.3 to 7.0 per cent. Plate LXXXI shows the degree of accuracy with which the diagrams and tables herein presented fit the great mass of experimental data shown on Plates LXXXVI*a* and LXXXVI*b*. It should be noted, however, that the Francis data shown on Fig. 8, Plate LXXXVI*a*, on which the Francis formula is based, are but an exceedingly small fraction of the accepted data. If it be argued that these diagrams do not give results which are absolutely correct, it cannot be denied

* "Lowell Hydraulic Experiments", pp. 134 and 135.

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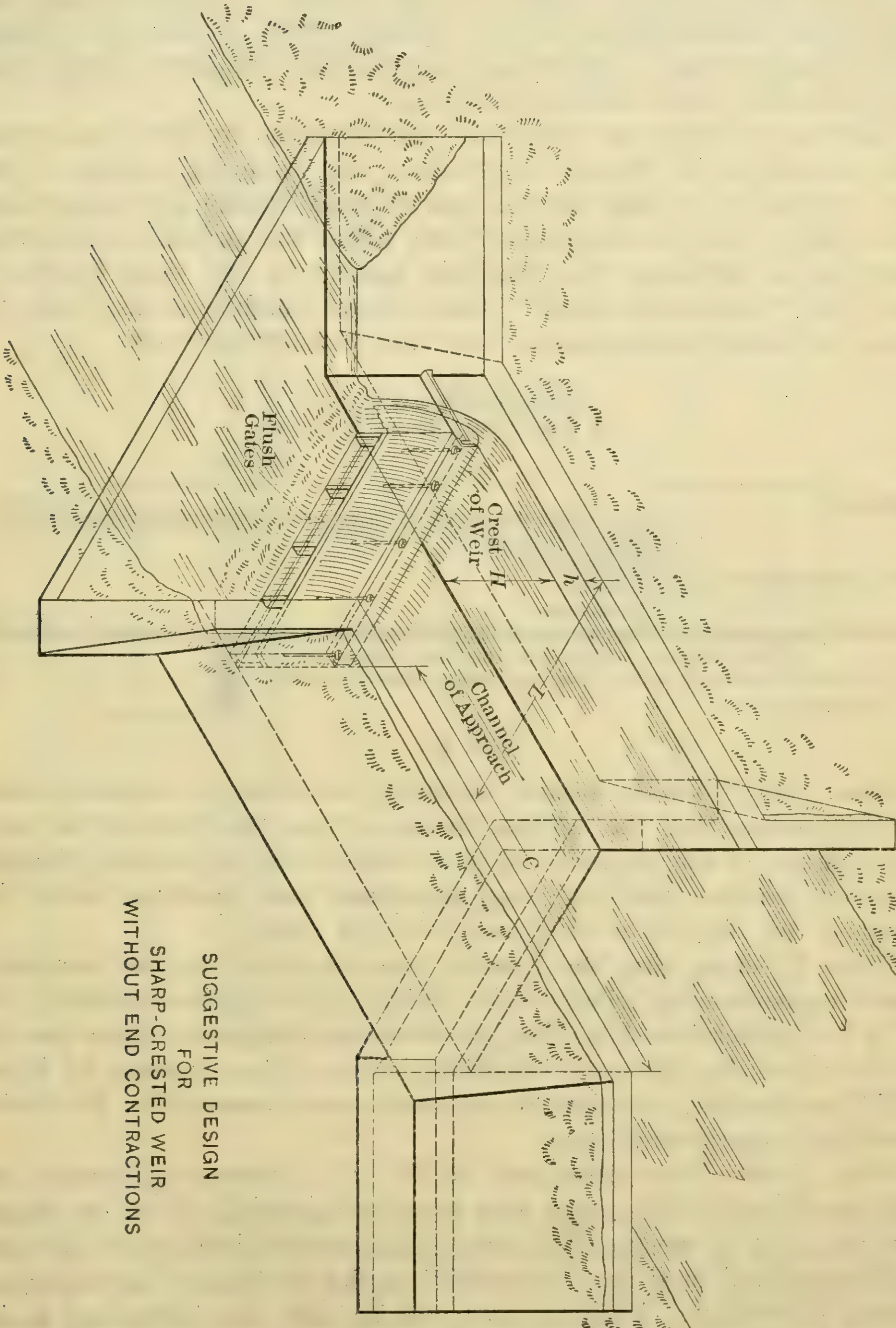

SUGGESTIVE DESIGN FOR SHARP-CRESTED WEIR WITHOUT END CONTRACTIONS

FIG. 50.

Mr. Lyman. that these results indicate positively that one right line or formula of the Francis form, namely, $Q = mh^n$ cannot be made to fit accurately these actual experimental results.

In the preparation of Table 72, the velocity of approach has not been given consideration, for two reasons: One is that it is truly, as Mr. Francis says, a "troublesome correction"; the other is that such corrections are rarely made in practice.

The figures below the heavy line are presented to show the greater errors given by the Francis formula if the "channel leading to the weir" has not "a suitable depth;" and it would hardly be exaggerating to say that in the mountainous West, where the streams carry great quantities of sediment, the channels leading to the weirs rarely, if ever, have "a suitable depth." Under these conditions, as Table 72 shows, discharge measurements vary from 8 to 24 per cent.

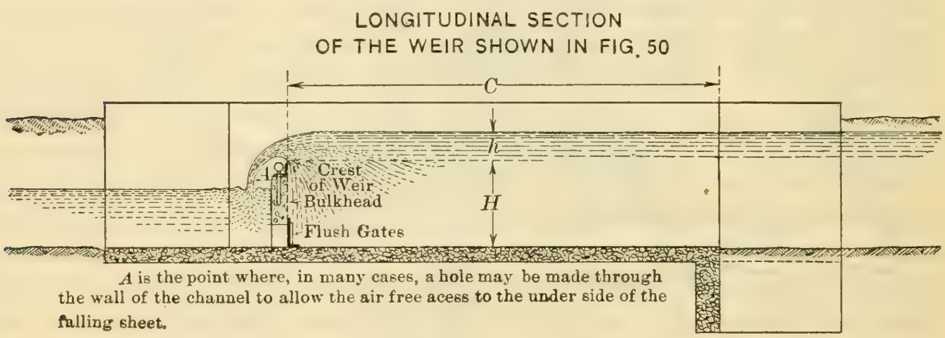

FIG. 51.

Some may think the figures presented are those of unusual cases. Table 72 shows that the vertical distance from the crest of the weir to the bottom of the channel is 6 in. or more in every instance, though, in actual practice, the untrained sometimes take readings when the bottom of the channel is on a level with the crest.

Table 72 shows that the Francis formula, when applied generally to weir measurements, gives results which are but approximations. Its use is the application of a right line where nothing but a curve can fit, and the curve that fits must be a flexible one. The diagrams on Plate LXXX are flexible, and they fit accurately every case within the limits of existing experimental data.

Why Heads Were Measured 15 Ft. Up Stream.—Although the heads considered in this paper were measured generally 15 ft. up stream from the crest by means of a "tape", this comparatively great distance was selected and used as a "standard" at the Hydraulic Laboratory of Cornell University. Very high as well as very low heads had to be measured with this same tape, and it was feared that at a less distance the "curved surface over the crest" might affect the readings for high heads. The cross-section of the channel of approach was so great, and

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the floor in it so nearly horizontal, that the results thus obtained are perhaps the same as if readings had been taken as Mr. Hazen suggests, at a distance up stream "equal to 2.5 times the height of the weir." However, unless some limit fixes the maximum allowable head on a weir of given height, perhaps it would be better to have the heads measured up stream from the crest a distance not less than some

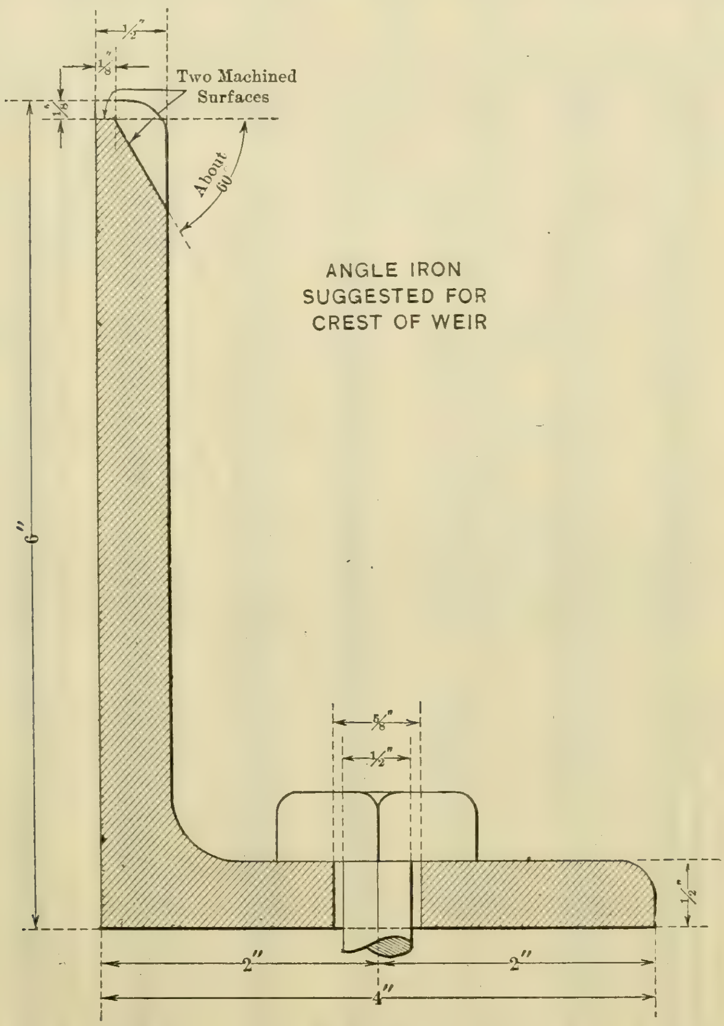

FIG. 52.

multiple of the maximum head to be allowed on the weir. Had the rule Mr. Hazen suggests been applied in the case of the upper standard weir in the Hydraulic Laboratory of Cornell University, the "tape" readings would have been taken up stream from the crest of the weir a distance of about 28 ft., because the weir is 11.25 ft. high.

Weirs Having Dimensions Proportional.—Concerning one other of the many excellent matters presented in the discussion of Mr. Hazen,

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TABLE 70.—Discharge, in Cubic Feet Per Second Per Foot of Length, Over Broad-Crested Weirs Without End Contractions.

These Quantities Were Read from Plate LXXXIII, Which Is Based on Experiments Made on Weirs 11.25 Ft. High in the Hydraulic Laboratory of Cornell University. For the Discharge over Weirs of Heights Other than 11.25 Ft., Apply the Coefficients Given in Table 0.

Head, in inches.	Head, in feet.	Weir 6 in. wide.	Weir 1 ft. 0 in. wide.	Weir 1 ft. 6 in. wide.	Weir 2 ft. 0 in. wide.	Weir 2 ft. 6 in. wide.	Weir 3 ft. 0 in. wide.	Weir 3 ft. 6 in. wide.
1 9/16	0.13	0.134	0.134	0.134	0.134	0.134	0.134	0.134
1 11/16	0.14	0.149	0.149	0.149	0.149	0.149	0.149	0.149
1 13/16	0.15	0.165	0.165	0.165	0.165	0.165	0.165	0.165
1 15/16	0.16	0.181	0.181	0.181	0.181	0.181	0.181	0.181
2 1/16	0.17	0.197	0.197	0.197	0.197	0.197	0.197	0.197
2 3/16	0.18	0.214	0.214	0.214	0.214	0.214	0.214	0.214
2 1/4	0.19	0.232	0.232	0.232	0.232	0.232	0.232	0.232
2 3/8	0.20	0.250	0.250	0.250	0.250	0.250	0.250	0.250
2 1/2	0.21	0.268	0.268	0.268	0.268	0.268	0.268	0.268
2 5/8	0.22	0.288	0.288	0.288	0.288	0.288	0.288	0.288
2 3/4	0.23	0.305	0.305	0.305	0.305	0.305	0.305	0.305
2 7/8	0.24	0.326	0.326	0.326	0.326	0.326	0.326	0.326
3	0.25	0.346	0.346	0.346	0.346	0.346	0.346	0.346
3 1/8	0.26	0.366	0.366	0.366	0.366	0.366	0.366	0.366
3 1/4	0.27	0.386	0.386	0.386	0.386	0.386	0.386	0.386
3 3/8	0.28	0.405	0.405	0.405	0.405	0.405	0.405	0.405
3 1/2	0.29	0.430	0.430	0.430	0.430	0.430	0.430	0.430
3 5/8	0.30	0.450	0.450	0.450	0.450	0.450	0.450	0.450
3 13/16	0.32	0.495	0.495	0.495	0.495	0.495	0.495	0.495
4 1/16	0.34	0.540	0.540	0.540	0.540	0.540	0.540	0.540
4 5/16	0.36	0.590	0.590	0.590	0.590	0.590	0.590	0.590
4 9/16	0.38	0.645	0.640	0.640	0.640	0.640	0.640	0.640
4 13/16	0.40	0.705	0.685	0.685	0.685	0.685	0.685	0.685
5 1/16	0.42	0.765	0.735	0.735	0.735	0.735	0.735	0.735
5 1/4	0.44	0.830	0.785	0.785	0.785	0.785	0.785	0.785
5 1/2	0.46	0.900	0.840	0.840	0.840	0.840	0.840	0.840
5 3/4	0.48	0.970	0.895	0.895	0.895	0.895	0.895	0.895
6	0.50	1.040	0.950	0.950	0.950	0.950	0.950	0.950
6 5/8	0.55	1.230	1.115	1.095	1.095	1.095	1.095	1.095
7 3/16	0.60	1.425	1.285	1.245	1.245	1.245	1.245	1.245
7 13/16	0.65	1.645	1.470	1.400	1.400	1.400	1.400	1.400
8 3/8	0.70	1.875	1.665	1.570	1.570	1.570	1.570	1.570
9	0.75	2.11	1.865	1.735	1.735	1.735	1.735	1.735
9 5/8	0.80	2.36	2.08	1.935	1.915	1.915	1.915	1.915
10 3/16	0.85	2.61	2.30	2.14	2.09	2.09	2.09	2.09
10 13/16	0.90	2.87	2.52	2.36	2.28	2.28	2.28	2.28
11 3/8	0.95	3.11	2.76	2.57	2.47	2.47	2.47	2.47
12	1.00	3.37	3.00	2.80	2.66	2.66	2.66	2.66
12 5/8	1.05	3.62	3.26	3.04	2.86	2.86	2.86	2.86
13 3/16	1.10	3.88	3.53	3.30	3.10	3.07	3.07	3.07
13 13/16	1.15	4.16	3.80	3.56	3.35	3.29	3.29	3.29
14 3/8	1.20	4.44	4.10	3.83	3.60	3.51	3.51	3.51
15	1.25	4.70	4.38	4.10	3.85	3.72	3.72	3.72
15 5/8	1.30	5.00	4.68	4.38	4.12	3.96	3.95	3.95
16 13/16	1.40	5.58	5.28	4.94	4.66	4.48	4.40	4.40
18	1.50	6.16	5.92	5.56	5.24	5.02	4.88	4.88
19 3/16	1.60	6.76	6.54	6.16	5.82	5.58	5.42	5.36
20 3/8	1.70	7.40	7.24	6.82	6.42	6.18	6.00	5.86
21 5/8	1.80	8.04	7.94	7.50	7.08	6.78	6.58	6.42
22 13/16	1.90	8.76	8.74	8.22	7.76	7.44	7.22	7.02
24	2.00	9.42	9.42	8.96	8.44	8.08	7.84	7.64
25 3/16	2.10	10.20	10.20	9.70	9.16	8.78	8.50	8.26
26 3/8	2.20	10.95	10.95	10.50	9.90	9.50	9.18	8.94
27 5/8	2.30	11.70	11.70	11.30	10.65	10.20	9.88	9.60
28 13/16	2.40	12.50	12.50	12.10	11.45	10.95	10.60	10.30
30	2.50	13.30	13.30	12.95	12.20	11.70	11.35	11.00

TABLE 70.—(*Continued.*)

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Head, in inches.	Head, in feet.	Weir 6 in. wide.	Weir 1 ft. 0 in. wide.	Weir 1 ft. 6 in. wide.	Weir 2 ft. 0 in. wide.	Weir 2 ft. 6 in. wide.	Weir 3 ft. 0 in. wide.	Weir 3 ft. 6 in. wide.
31 3/16	2.60	14.15	14.15	13.85	13.10	12.55	12.10	11.80
32 3/8	2.70	15.00	15.00	14.75	13.95	13.35	12.90	12.55
33 5/8	2.80	15.90	15.90	15.70	14.80	14.15	13.70	13.30
34 13/16	2.90	16.75	16.75	16.60	15.70	15.00	14.50	14.05
36	3.00	17.65	17.65	17.60	16.60	15.90	15.35	14.90
38 3/8	3.20	19.50	19.50	19.50	18.50	17.70	17.05	16.55
40 13/16	3.40	21.5	21.5	21.5	20.5	19.60	18.90	18.25
43 3/16	3.60	23.5	23.5	23.5	22.6	21.6	20.8	20.1
45 5/8	3.80	25.4	25.4	25.4	24.7	23.6	22.7	22.0
48	4.00	27.6	27.6	27.6	26.9	25.8	24.8	24.0
50 3/8	4.20	29.7	29.7	29.7	29.2	27.9	26.8	25.8
52 13/16	4.40	31.8	31.8	31.8	31.5	30.0	28.9	27.8
55 3/16	4.60	34.2	34.2	34.2	34.0	32.4	31.1	30.0
57 5/8	4.80	36.6	36.6	36.6	36.5	34.9	33.5	32.2
60	5.00	38.8	38.8	38.8	38.8	37.3	35.8	34.5

TABLE 71.—SUGGESTIVE DIMENSIONS FOR WEIRS TO BE USED FOR MEASURING VARIOUS QUANTITIES OF FLOWING WATER.

Further Information, Concerning the Meaning of the Notation Used, is Given on Figs. 50 and 51.

Minimum, Q, in cubic feet.	Maximum, Q, in cubic feet.	h, in feet.	H, in feet.	Height of wall, in feet.	L, length of weir, in feet.	C, length of channel, in feet.	Thickness of floor, in inches.	Thickness of bottom of wall, in inches.	Thickness of top of wall, in inches.	Length of wing wall, in feet.	Depth of cut-off wall, in feet.	Concrete, in cubic yards.
0.05	0.25	0.5	0.5	1.0	0.2	2	6	8	6	2.0	1.0	0.5
0.15	0.60	0.5	0.5	1.0	0.5	4	6	8	6	2.0	1.0	0.7
0.30	1.30	0.5	0.5	1.0	1.0	6	6	8	6	2.0	1.0	0.9
0.45	1.90	0.5	0.75	1.25	1.5	8	6	8	6	2.0	1.0	1.4
0.60	2.50	0.5	0.75	1.25	2.0	10	6	8	6	2.5	1.0	2.2
0.75	3.2	0.5	0.75	1.25	2.5	10	6	8	6	2.5	1.0	2.3
1.0	3.7	0.5	1.0	1.5	3.0	10	6	8	6	2.5	1.0	2.4
1.5	11.0	1.0	1.0	2.0	3.5	12	6	8	6	3.0	1.0	3.5
2.0	15.0	1.0	1.0	2.0	4.0	12	6	8	6	3.0	1.0	3.7
2.5	18.0	1.0	1.5	2.5	5.0	12	6	8	6	3.0	1.0	4.0
3.0	21.0	1.0	1.5	2.5	6.0	14	8	12	6	3.0	1.5	5.7
3.5	25.0	1.0	1.5	2.5	7.0	14	8	12	6	4.0	1.5	6.2
4.0	28.0	1.0	2.0	3.0	8.0	14	8	12	6	5.0	1.5	10.0
4.5	32.0	1.0	2.0	3.0	9.0	16	8	12	8	5.0	1.5	11.4
5.0	65.0	1.5	2.0	3.5	10.0	16	8	12	8	8.0	2.0	18.0
6.0	71.0	1.5	2.0	3.5	11.0	16	8	12	8	8.0	2.0	18.7
7.0	78.0	1.5	2.0	3.5	12.0	16	12	15	8	8.0	2.0	24.5
8.0	84.0	1.5	2.0	3.5	13.0	16	12	15	8	8.0	2.0	25.2
9.0	91.0	1.5	3.0	4.5	14.0	18	12	15	8	9.0	2.5	34.6
10.0	97.0	1.5	3.0	4.5	15.0	18	12	18	12	10.0	3.0	44.6
11.0	104.0	1.5	3.0	4.5	16.0	18	12	18	12	10.0	3.0	47.5
12.0	170.0	2.0	3.0	5.0	17.0	18	12	18	12	10.0	3.0	48.7
13.0	180.0	2.0	3.0	5.0	18.0	18	12	18	12	10.0	3.0	49.9
14.0	200.0	2.0	3.0	5.0	20.0	20	12	18	12	10.0	3.0	55.5
15.0	215.0	2.0	4.0	6.0	22.0	20	12	24	12	12.0	4.0	78.0
16.0	230.0	2.0	4.0	6.0	24.0	20	12	24	12	12.0	4.0	80.8
17.0	250.0	2.0	4.0	6.0	26.0	20	12	24	12	12.0	4.0	83.6
18.0	270.0	2.0	4.0	6.0	28.0	20	12	24	12	12.0	4.0	86.4
19.0	290.0	2.0	4.0	6.0	30.0	20	12	24	12	12.0	4.0	89.2
20.0	310.0	2.0	4.0	6.0	32.0	20	12	24	12	12.0	4.0	92.2

Mr. Lyman. *viz.*, "that weirs which, in all respects, are proportional to each other will have the same coefficients", the writer wishes to say that for years he has attempted to get into clear form this same idea. Now that this principle has been clearly stated, he will attempt to demonstrate in the Hydraulic Laboratory of the University of Utah whether or not it is correct.

TABLE 72.—PERCENTAGE OF DIFFERENCE BETWEEN QUANTITIES OF DISCHARGE OVER WEIRS OF VARIOUS HEIGHTS, AS GIVEN BY PLATE LXXX, AND AS COMPUTED BY THE FRANCIS FORMULA: $Q_F = 3.33\,H^{\frac{3}{2}}$.

Q_F = Discharge, in cubic feet per second.

H = Height of weir, in feet.

The quantities above the heavy line are within the limits for the formula specified by Mr. Francis.

Head, in feet.	Q_F	QUANTITY, AS READ FROM PLATE LXXX.							
		$H = 0.5$ ft.	$H = 0.75$ ft.	$H = 1.0$ ft.	$H = 1.5$ ft.	$H = 2.0$ ft.	$H = 3.0$ ft.	$H = 4.0$ ft.	$H = 6.0$ ft.
0.2	0.298	0.315 5.4%	0.314 5.1%	0.313 4.8%	0.312 4.5%	0.311 4.2%	0.310 3.9%	0.309 3.6%	
0.3	0.547	0.595 8.1%	0.584 6.3%	0.576 5.0%	0.570 4.0%	0.566 3.4%	0.563 2.8%	0.560 2.3%	
0.4	0.842	0.930 9.5%	0.905 7.0%	0.893 5.7%	0.875 3.8%	0.870 3.2%	0.860 2.1%	0.855 1.5%	0.850 0.9%
0.7	1.950	2.295 15.0%	2.180 10.5%	2.130 8.5%	2.070 5.8%	2.030 3.9%	1.995 2.3%	1.975 1.3%	1.965 0.8%
1.0	3.330	4.230 21.3%	3.900 14.6%	3.780 11.9%	3.640 8.5%	3.555 6.3%	3.440 3.2%	3.400 2.1%	3.375 1.6%
1.3	4.936	6.51 24.2%	5.975 17.4%	5.775 14.5%	5.525 10.7%	5.350 7.7%	5.150 4.2%	5.050 2.3%	5.020 1.7%
1.6	6.739	....	8.40 19.8%	8.07 16.5%	7.69 12.4%	7.41 9.2%	7.08 4.8%	6.94 3.0%	6.89 2.3%

The Weir Without End Contractions Gives the Most Accurate Results.—Mr. Moritz says the statement that the "sharp-crested weir without end contractions" gives "the most accurate results with the smallest amount of work * * * is open to serious question", as the use of such weirs "would be practically impossible on account of the large quantity of silt carried at certain seasons of the year" in the canals of some irrigation projects.

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Great Velocity of Approach and Little Sediment Deposited.—The first of the six reasons why these weirs are best, given near the beginning of this paper, was that, with this structure, the velocity of the water above the weir is greatest, and therefore its capacity for carrying silt over the crest is also greatest. The second of the six reasons is that, with this weir, the channel of approach must be made of concrete, lumber, or some other material, that will retain a definite rectangular form; it is easy, therefore, to determine whether or not the channel is clean, and it must be clean to give good results. Another reason may now be added: This weir is best because its crest may be fixed in position permanently, and flush-gates may be built under it. Through these gates silt may pass when measurements are not actually being taken. Thus is maintained a relatively great velocity in this comparatively small channel of approach, and the channel is kept clean.

The Diagrams and Tables Fit Great Variety of Weirs and Great Variation of Heads.—Again, as previously explained, the diagrams and tables submitted in this paper (Plate LXXX and Table 69) fit, and fit accurately, a great variety of weirs of this type, and do so for a comparatively large range of heads. The Francis diagram or formula for suppressed weirs, based on only 17 experiments, cannot be expected to fit, nor does it fit, the great variety of weirs and the large range of heads to which it has been and is being applied. The smallest head used by Mr. Francis was greater than 0.73 ft., and the largest less than 1.1 ft.; his experiments, therefore, cover a difference in head of but little more than 3 in. Moreover, all his experiments were made on only one weir with only one height.

Francis Formula Fits Within Very Narrow Limits.—The Francis experiments are shown on Fig. 8, Plate LXXXVI*a*. The results obtained by other experiments are presented on both sides of this figure. They indicate that if Francis had used higher and lower heads, he would have discovered that no one equation of the form he uses ($Q = mh^n$) can fit accurately a series of experiments on weirs of the type herein considered.

The Best Practical Measuring Device Available.—Mr. Moritz closes his argument in favor of the Cippoletti weir with these words:

> "In the writer's opinion, there is no better practical device, at present available, than the Cippoletti weir for measuring the quantities encountered in the majority of irrigation laterals."

The velocity of the water approaching the Cippoletti weir is less than that of the water approaching the weir without end contractions. In order to give accurate results, the cross-section of the channel of approach for the former must be greater than that for the latter. The deposit of silt, therefore, will be greater above the Cippoletti weir than above the weir without end contractions. If, then, with the Cippoletti

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weir, as he explains, "one or two good cleanings of the pool during the season will generally suffice", less cleaning will be required for the weir without end contractions. Still more important is the fact that a conscientious water master may clean his canal so as to make a large pool, and a water master of the other sort may make but a small pool, or none at all. Table 72 shows what effect a change in the cross-section of the "pool" or channel of approach has on the discharge. In the case of the suppressed weir, the channel to be cleaned has a fixed cross-section, of concrete, timber, or other material, that will retain its definite rectangular section. With sluice-gates through the bulkhead under the crest of the weir, this channel can be cleaned with the aid of the water much more easily than can the "pool" of which Mr. Moritz speaks.

If the channel above the suppressed weir has 6 in. of silt in it, the discharge (by taking this fact into account) can be found from the diagrams and tables herewith submitted. This cannot be done with the Cippoletti weir.

The writer does not see wherein the use of the tables and diagrams recommended in this paper requires any greater care than does the use of the Cippoletti weir and the diagram Mr. Moritz submits. Neither does he concede that the Cippoletti weir is the simpler of the two, in the sense of its being easier to use; but he will concede that it may be installed at a little less cost.

Mr. Moritz does not give a reference to the experiments which he states show that the Cippoletti weir gives results "within an error of from 2 to 4% when the head is not greater than one-third of the crest length." However, as the Francis formula is used for computing the discharge over the Cippoletti weir, it is important to specify the distance of each contraction from the sides and from the bottom of the channel. Of course, the greater the cross-section of the channel of approach, the more nearly will conditions approach the ideal; and the nearer this ideal is approached, the greater will be the tendency to fill the channel with silt.

Accuracy of Results.—Mr. Moritz claims that the Cippoletti weir, under what he intimates are rather limited conditions, gives results "within an error of from 2 to 4%"; but he fails to give the quantity or value of the data on which he bases this assertion. He does explain, however, that "the quantities in these experiments were measured volumetrically", and intimates in the same sentence that there are errors in the tables and diagrams herein presented "due to the use of a calibrated measuring device * * * such as the 'standard weir' used in the Cornell Experiments," giving results which may be "accepted as accurate to within 2 or 3 per cent."

Evidently, Mr. Moritz has gone carefully over the matter presented in this paper, but he did not see for the moment that the "2 or 3 per

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cent." applies to "Broad-Crested Weirs" (page 1523*), and not to "Sharp-Crested Weirs Without End Contractions" (page 1516*). It is for the latter weirs and for the diagrams and tables which accompany them that this paper makes its greatest claim. The results are based on the twenty "classic" series of experiments named on the left of Plate LXXXI. This plate indicates clearly with what degree of accuracy the diagrams and tables of this paper fit this unparalleled array of hydraulic data.†

Discrepancies in Quantities for Weirs with Broad Crests.—To furnish the "additional light" for which Mr. Moritz asks concerning certain runs on broad-crested weirs, Fig. 53 has been made. The curves shown are drawn to fit the points based on the original experiments. These points are represented by large dots. The results reached by Mr. Williams‡ are shown by small crosses, and those obtained by Mr. Horton§ are represented by small dots within triangles.

The accuracy with which the results given by the diagrams on Plates LXXXV and XC fit those obtained by using the original data is shown by the circles on Fig. 53. This information and the original data on which Messrs. Williams and Horton also based their results are given in Table 73. From the quantities given, and their signs, in the column marked "Percentage of difference", it is seen that the results obtained by using Plates LXXXV and XC fit the original experiments almost perfectly. There are only five individual experiments which show a difference greater than 2%, and two of these would show still smaller differences but for the fact that slight errors in calculation were made in the original computations.

Accuracy with Which Diagrams Fit Original Data.—Attention is again drawn to Plates LXXXIX and XCI*a*. These plates show that the diagrams giving mean values furnish results which can be used with greater confidence than can the results of the individual runs. Other plates in this paper give the same information concerning the accuracy of the other diagrams.

Although the Suppressed Weir Is Recommended as the Best Weir, Other Measuring Devices Are Necessary.—It was not, as Mr. Johnston seems to have understood, the writer's intention that the weir be adopted by legislation as the only device for measuring water. The intention was to advocate that, by legislation, the weir without end contractions be adopted as the "standard weir". It can be used to obtain accurate results wherever any other type of weir can be used, and in many places and under many conditions where reliance can be placed on no other weir. If, as Mr. Robinson says, this type of weir

* *Proceedings*, Am. Soc. C. E., for September, 1913.

† Reference is given in the proper places in this paper to the original sources from which these data came.

‡ "American Civil Engineers Pocket Book," p. 869.

§ U. S. Geological Survey, Water Supply Paper No. 200, pp. 190 and 192.

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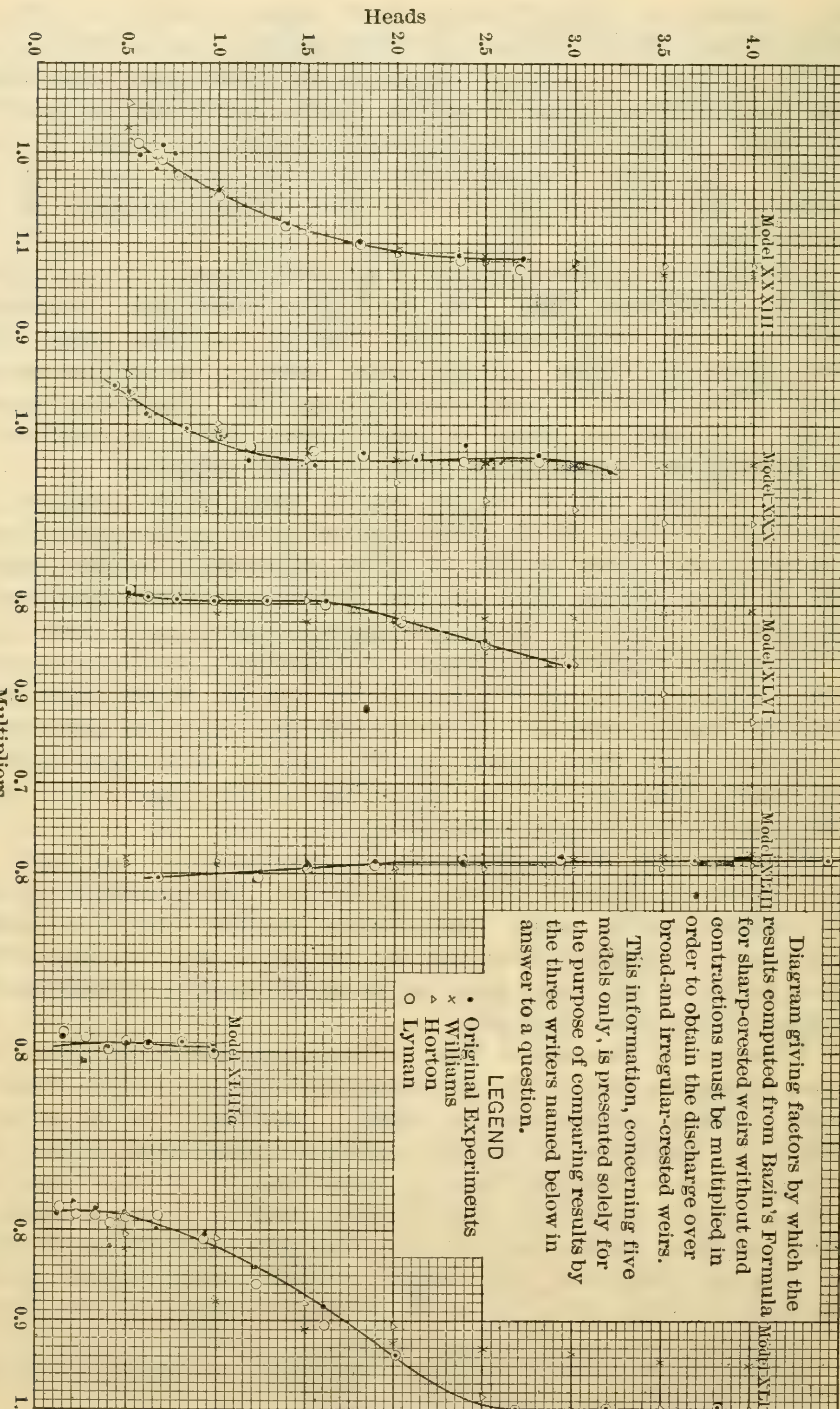

Multipliers

Fig. 53.

were made standard by Federal Statute, then it would soon be used universally. By further actual experiment on this one weir, if the results based on the data at present available are not absolutely correct, data will be obtained from which more and more accurate results can be found. Mr. Lyman.

With this suggested standard weir, it is believed, gravel and silt can be disposed of, as already explained, without constructing the weir in a channel parallel to the main canal, as Mr. Johnston suggests.

Basic Theory on Which Diagrams Are Drawn.—As it was the intention to give the results of this investigation first and the theory afterward, it is not surprising that Mr. Johnston, in his discussion, states that the basic theory of the method used is not explained. He also says, after stating that the differences used cannot be shown with sufficient accuracy, when the rectangular axes are used, "it would seem to one who has not given extended study to the problem that Mr. Lyman continues to use rectangular axes."

On all the plates intended for giving discharges, it may be seen that the lines representing heads and those representing discharges are not at right angles to one another.

The beginning of the explanation of the graphic method used in the construction of these diagrams is on page 1534, headed "E.—Method of Determining the Proposed Formulas." Unfortunately, on page 1545 a reference to "Fig. 2, Plate LXXXVI*a* (right hand end)" should be to "Fig. A, Plate LXXXVI*b* (right hand end)". Further explanation of the theory on which the construction of these diagrams is based, and particularly that portion relating to the inclination of the axes is given beginning on page 1566 in the part entitled "I.—Method of Constructing the Final Diagrams."

Although Mr. Johnston's "basic theory" is certainly ingenious, and of intense interest from the mathematical point of view, it could perhaps not be applied so easily to the construction of the diagrams herein given as can the method, largely graphical, which was actually used in their construction.

The Results Fit Perfectly the Conditions Prevailing in Ordinary Irrigation Streams.—Mr. Winsor states, with respect to the experiments made on weirs without end contractions:

"Most of the observations thus far have been conducted under conditions quite foreign to those which are met in the operation of a canal system. The early experimenters used, in the main, larger streams than the ordinary irrigation stream requiring measurements, and the recent experiments were conducted under laboratory conditions which are seldom applicable to the ordinary irrigation stream."

The heads presented in this paper were measured with a "tape", or else have been reduced to equivalent "tape readings"; heads can be thus conveniently measured in any canal. Mr. Winsor did not suggest a simpler or more practical method. If the water in the canal system

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TABLE 73.—DIFFERENCES IN PERCENTAGE BETWEEN DISCHARGES OVER BROAD-CRESTED WEIRS AS ACTUALLY MEASURED AND AS READ FROM PLATES LXXXV AND XC; ALSO FACTORS BY WHICH THE RESULTS COMPUTED FROM BAZIN'S FORMULA FOR SHARP-CRESTED WEIRS WITHOUT END CONTRACTIONS MUST BE MULTIPLIED IN ORDER TO OBTAIN THE DISCHARGE OVER BROAD-CRESTED AND IRREGULAR-CRESTED WEIRS.

Q_m = Discharge, in Second-Feet, as Actually Measured.

Q_L = Discharge, in Second-Feet, as Read from Plates LXXXV and XC.

Q_B = Discharge, in Second-Feet, for Sharp-Crested Weirs, as Computed by Bazin's Formula.

$$M_m = \frac{Q_m}{Q_B}.$$

$$M_L = \frac{Q_L}{Q_B}.$$

MODEL XXXIII.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
2.660	16.272	16.45	− 1.09	14.55	1.118	1.130
2.352	13.464	13.55	− 0.64	12.07	1.116	1.123
1.784	8.747	8.76	− 0.15	7.94	1.101	1.102
1.394	5.914	5.94	− 0.44	5.475	1.079	1.085
0.992	3.439	3.460	− 0.61	3.300	1.042	1.048
0.752	2.197	2.240	− 1.96	2.192	1.001	1.022
0.679	1.866	1.896	− 1.61	1.882	0.992	1.007
0.653	1.808	1.785	+ 1.27	1.775	1.018	1.004
0.553	1.397	1.375	+ 1.57	1.389	1.006	0.992

MODEL XXX.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
3.187	20.152	20.00	+ 0.75	19.17	1.051	1.043
2.790	16.219	16.42	− 1.24	15.65	1.037	1.049
2.384	12.631	12.88	− 1.97	12.35	1.022	1.042
2.094	10.577	10.55	− 0.26	10.15	1.041	1.039
1.793	8.282	8.30	− 0.22	8.00	1.034	1.037
1.532	6.607	6.504	+ 1.56	6.317	1.045	1.030
1.174	4.399	4.350	+ 1.11	4.238	1.039	1.027
1.010	3.436	3.430	+ 0.18	3.390	1.012	1.011
0.802	2.409	2.416	− 0.29	2.409	1.000	1.002
0.613	1.608	1.600	+ 0.48	1.621	0.992	0.997
0.508	1.195	1.200	− 0.42	1.239	0.966	0.970
0.427	0.921	0.920	+ 0.11	0.960	0.959	0.958

MODEL XLVI.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
2.965	14.901	14.85	+ 0.34	17.18	0.868	0.865
2.486	11.032	11.13	− 0.89	13.13	0.841	0.848
2.030	7.895	7.95	− 0.70	9.68	0.816	0.822
1.597	5.360	5.38	− 0.37	6.711	0.798	0.803
1.232	3.628	3.624	+ 0.11	4.556	0.797	0.799
0.972	2.549	2.549	± 0.00	3.201	0.797	0.798
0.784	1.856	1.857	− 0.05	2.328	0.797	0.797
0.602	1.255	1.256	− 0.08	1.578	0.796	0.796
0.503	0.967	0.963	+ 0.41	1.222	0.791	0.788

TABLE 73.—(*Continued.*)

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MODEL XLIII.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
4.432	25.011	24.94	+ 0.28	31.95	0.783	0.781
3.661	18.651	18.69	− 0.21	23.78	0.785	0.786
2.935	13.200	13.27	− 0.53	16.94	0.779	0.783
2.360	9.608	9.54	+ 0.71	12.15	0.791	0.784
1.890	6.841	6.87	− 0.42	8.66	0.789	0.794
1.480	4.767	4.78	− 0.06	6.012	0.793	0.796
1.206	3.520	3.550	− 0.85	4.413	0.798	0.805
0.689	1.544	1.545	− 0.07	1.924	0.803	0.804

MODEL XLIIIa.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
0.986	2.618	2.625	− 0.27	3.270	0.800	0.803
0.981	2.568	2.605	− 1.44	3.245	0.791	0.803
0.786	1.847	1.850	− 0.16	2.337	0.791	0.793
0.621	1.309	1.310	− 0.08	1.653	0.792	0.793
0.496	0.945	0.942	+ 0.32	1.196	0.790	0.788
0.392	0.671	0.674	− 0.45	0.844	0.795	0.798
0.279	0.421	0.409	+ 2.85	0.520	0.810	0.787
0.161	0.184	0.185	− 1.09	0.238	0.774	0.778

MODEL XLI.

Head, in feet.	Q_m	Q_L	Percentage of difference.	Q_B	M_m	M_L
3.841	25.531	25.47	+ 0.24	25.59	0.999	0.999
3.176	18.995	19.04	− 0.24	19.06	0.998	0.999
2.674	14.624	14.36	+ 1.80	14.66	0.999	0.999
2.021	9.021	9.00	+ 0.23	9.61	0.938	0.937
1.601	5.936	6.08	− 2.42	6.72	0.883	0.905
1.233	3.835	3.920	− 2.21	4.562	0.841	0.860
0.941	2.476	2.485	− 0.36	3.058	0.810	0.814
0.670	1.472	1.442	+ 2.04	1.845	0.798	0.782
0.488	0.911	0.917	− 0.66	1.167	0.781	0.786
0.417	0.759	0.735	+ 3.16	0.926	0.819	0.794
0.330	0.520	0.526	− 1.15	0.669	0.777	0.787
0.210	0.273	0.278	− 1.83	0.355	0.769	0.783
0.122	0.130	0.128	+ 1.54	0.166	0.784	0.772

is measured with any degree of care, then, with the heads measured as above, the conditions under which these experiments were made appear to be identical with those "met in the operation of a canal system."

We may wait long for many series of experiments to be made with such care that they will deserve a place among those used in this paper. It will certainly be a much longer time before future experiments will warrant disregarding these, even if the results they give are not "absolutely correct."

As to the quantity of water actually measured: Plate LXXXI gives the list of the experiments on which Plate LXXX is based. For the results given by Plate LXXX, some of which are presented in Table 69, a high degree of accuracy is claimed. These experimental data cover the limits given both on Plate LXXX and in Table 69. The

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writer is confident that they will give accurate results under conditions "which are met in the operation of a canal system."

Little Additional Cost Brings Better and More Accurate Results.—The small additional cost of "constructing a channel of approach", instead of making "the enlargement of the up-stream portion of the stream bed to a sufficient area" (and Mr. Winsor might well have added to a sufficient depth), will, in the opinion of the writer, be much more than compensated by the additional convenience, accuracy, and satisfaction that will result. Mr. Winsor certainly appreciates the value of maintaining a large velocity of approach in this mountainous country, as does every other engineer who has had experience where streams carry great quantities of silt.

Information Concerning One Particular Weir More Needed Now Than Information Concerning a General Solution.—It is not clear why Mr. King thinks that, in order to be of general application, the solution to the weir problem "must provide for a channel of approach of any cross-section." Of course, conditions will arise which make it desirable to know what quantity of water with a given head is passing over some irregular crest having uncertain contractions. Great care has been taken and much space used to provide, in this paper, a means of measuring the quantity of water flowing over irregular weirs, and the results reached are satisfactory; but, when water is to be measured with a high degree of accuracy, the device to be used must be built for this purpose.

As the suppressed weir can be used under practically all conditions where accurate results are demanded, the writer thinks it better for hydraulicians, who are in a position to do so, to gather more information concerning the discharge over this weir than to attempt to get a "satisfactory general method of solution for all weir problems." Although this general information would be of interest and value, the writer regards the more scientific data as having a greater value.

The Greater the Velocity of Approach the Better.—As water rarely if ever flows over a weir without some velocity of approach, Mr. King's suggestion to secure data for this condition does not appeal to the writer as of great importance. In fact, as the greater the velocity of approach the more ideal is the weir for use where silt is carried in the water, would it not be better to gather information concerning weirs with smaller heights and therefore with still greater velocities of approach?

The Best Standard Weir for All Cases.—Taking into account the quantity and reliability of the data available, the range of heights covered by the series of weir experiments used herein (see list on Plate LXXXI), and the comparatively wide range of heads covered by these experiments, the writer must differ with Mr. Horton, and say that there is a weir "which is the best standard for all cases", and that it

should be so considered by all engineers until data are furnished that will show some other weir at least equal, in its general application, to that without end contractions. Mr. Lyman.

Simplicity of Method.—Mr. Horton's discussion argues in favor of making "the computation as simple as possible by using the $\frac{3}{2}$ power of the head, instead of causing the head to vary according to some other fractional or decimal power, which would require the use of logarithms, logarithmic paper, or a logarithmic slide-rule for practical computations," and he closes remarking "regretfully, as regards hydraulic science, 'Of the making of many formulas there is no end'".

Although this paper presents a great array of formulas, these are used, and are intended only to be used, for the purpose of preparing diagrams similar to those shown on Plate LXXX and in Table 69, from which, without calculation, discharges are obtained.

Mr. Smith states that he cannot agree that the diagrams herein presented are "in the interests of simplicity." Then he adds "errors in taking results from the diagrams can easily exceed the discrepancy between the author's and the Francis formulas." The writer believes that Mr. Smith will change his view after examining Table 72. If he takes the results obtained from Plate LXXX and checks them by Table 69, he will probably be surprised at the smallness of the "discrepancy."

Discharge Proportional to Length of Crest.—Mr. Smith says, again, "one assumption made at the outset of the paper is perhaps open to question. It is that the discharge is proportional to the length of crest."

In reply, attention is drawn to the fact that the famous hydraulician, Bazin, made very careful experiments on his two standard weirs, one having a length of 1 m., the other having a length of 2 m. Concerning the results obtained, he writes:

"All needful precautions were taken to insure a flow under exactly the same conditions as over the 2-m. weir. The height of the weir remained the same. It might, however, be asked whether, in view of a somewhat different distribution of the velocities, or of an increase in the effect of friction against the side-walls, the results would be strictly comparable. We have failed to observe any appreciable difference between the two series."

In the new canals of the Hydraulic Laboratory of the University of Utah, the writer hopes to be able to determine whether or not the discharge is proportional to the length of crest, and also if the friction on the side-walls, where the velocity is comparatively small, is a factor that must be taken into account when a high degree of accuracy is desired.

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Mr. Smith asserts, with others, that "the Cippoletti weir discharge is proportional to the length." If this were true in the sense that the same quantity of water passes over each and every unit of length of crest of this weir, it could be used for a dividing device, as can the weir without end contractions. A moment's reflection shows that water flows through the triangular areas outside of the ends of the crest proper; it follows, therefore, that the discharge is not proportional to the length. For this reason the Cippoletti weir cannot be used as an accurate dividing device.

Simplicity of Table 69.—The table of discharges (Table 69) is believed to be quite as simple as the table referred to by Mr. Smith, which he says "is so simple that an engineer is not required to interpret it."

Height of Crest of Cippoletti Weir Omitted.—If the "height" of the crest of the Cippoletti weir above the bottom of the channel of approach is not specified, then the statement made by Mr. Smith that the error, under the three conditions he names, "will be less than 1%" is far from correct.

One Type of Weir Should Be Established by Legislation.—"Exception," Mr. Smith says, "should be taken" to the "proposition to establish one type of weir for universal use by legislation." Every engineer who has been connected with power-plant work, involving, as it does and must, the efficiency of hydraulic motors, has learned that, in order to determine in a satisfactory way the quantity of water used, it is wise, if not positively necessary, to specify how the water is to be measured. When, as is the case these days, "bonuses and forfeits" involving thousands of dollars are based on variations of discharge as small as one-tenth of 1%, as stated by Mr. Williams, it is certainly advisable, by legislation or otherwise, to fix some one standard form of weir measurement. In such a case it is as important, perhaps, to get uniform results as to get results which are accurate.

Mr. Smith says, "The best method for a particular case depends on the surrounding conditions." The writer holds that the best method for all cases is that by which the same quantity of flowing water can be measured again and again by different trained observers, each measurement giving the same result. This is especially true if the result is an accurate one.

That the Utah law establishing uniform cross-sections for roads is an unwise one is a fact that has no bearing on this discussion.

Results Inaccurate When not Based on Experiment.—Can Mr. Smith know, or can he or any one else find out, at any reasonable cost, what degree of accuracy he obtained when measuring water with the special weir described in his discussion? The channel of approach, he says, was "scoured out to a depth of from 1 to 2 ft." Thus great additional uncertainty was introduced into the results.

Modified Venturi Meter.—The writer agrees with Mr. Smith that "in the western part of the United States it is likely that a modification of the Venturi water meter will come into use to a considerable extent." Experimental data on a device of this sort will be welcomed. Mr. Lyman

Undershot Weir.—Another device, which may be used with great satisfaction in the near future, is the "inverted weir" without end contractions, that is, a notch in the bottom of a diaphragm across the channel, with contraction at the top only. Through such a notch or under such a "weir", sediment would pass readily. In order to make this structure a very valuable measuring device, it only remains for some one to determine the relation between the quantity of water flowing through the notch and the difference in elevation between the surface of the water above and the surface of the water below the diaphragm.

The Cippoletti or the Suppressed Weir?—Mr. Jarvis writes:

"If a single type of weir were adopted, either the Cippoletti or the sharp-crested rectangular weir without end contractions would doubtless serve best in irrigation practice."

For accurate work, where measurements are to be taken over comparatively long periods and under varying conditions, practically every good reason is in favor of the suppressed and against the Cippoletti weir.

Weir for Quick Rough Work.—A case cited by Mr. Jarvis is a good one in favor of the Cippoletti weir; that is, where a portable device of this sort is used by a ditch rider. When such rough and hurried measurements give the accuracy desired, it may be wise to use the Cippoletti weir.

Measuring Flume and Current Meter.—Mr. Jarvis draws attention to the use of the measuring flume and the current meter. In canals where the slope is slight, these can be wisely used.

Maximum Discharge, Minimum Obstruction.—The writer is grateful to Mr. Jarvis for the clear statement that "the rectangular weir without end contractions provides the maximum width of crest, and the minimum obstruction to the flow in the canal."

Difficulties Encountered in Accurate Measurement of Heads.—The value of this paper is greatly increased by the keen comments of Professor Williams. He points out errors which are introduced into weir work through inaccuracies in measuring heads. He also presents much original matter relating to differences which occur when the same head is measured on the same weir at the same time by different persons and in different ways. He points out specifically the difficulties encountered when an attempt is made to measure heads carefully.

Conclusion.—In conclusion the writer could hardly do better, perhaps, than refer to the six reasons given near the beginning of this

Mr. Lyman. paper setting forth the advantages in practice of using weirs without end contractions. In the language of Mr. Jarvis, these weirs "provide the maximum width of crest and the minimum obstruction to the flow in the canal." This is a vital matter where streams carry great quantities of silt.

New practical features added since the paper was written are the use of flush-gates in the bulkhead which carries the permanently fixed crest (Figs. 50 and 51); suggestive definite dimensions for weirs to be used for measuring certain quantities of water, and the quantities of concrete required to build the structures recommended (Table 71); and a table giving discharges over all sharp-crested weirs without end contractions within the limits of experimental data (Table 69).

The writer is more strongly convinced than ever that when irrigation engineers and practical hydraulicians have given careful consideration to the matters contained herein they will agree with him that, for general application, where accurate practical information is wanted, and where any weir can be used for measuring water, that without end contractions will answer the purpose best.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TOPOGRAPHICAL SURVEYS MADE BY THE AMERICAN SECTION OF THE INTERNATIONAL BOUNDARY COMMISSION UNITED STATES AND MEXICO.

Discussion.*

By W. W. Follett, M. Am. Soc. C. E.†

W. W. Follett, M. Am. Soc. C. E. (by letter).—Owing to illness and absence from his office, where all his records are kept, the writer can only review the discussion of this paper briefly. The interest shown by the members in the subject has been gratifying. Mr. Follett.

Mr. Landreth brings out quite plainly the relation between the number of shots per square mile and the cost. His figures, however, as would be expected, show that the cost does not increase in direct ratio to the number of shots. With 15 times as many shots as on the work described by the writer, his cost was only 4½ times as much per square mile. Of course, conditions as to brush, etc., may have affected the relative cost of the two pieces of work.

After further study, the writer is inclined to agree with Mr. Brown that he set the maximum errors too high, and will modify his statement on that subject to read:

"In any work where a variation of ½ m. to 1 m. in the relative location of points near together, or 3 or 4 m. in that of those which are material distances apart, can be tolerated, the stadia offers a most rapid and handy method of work."

Beyond this, he cannot go. The Cincinnati surveys must have been most thoroughly controlled by triangulation. Even then, the writer does not see how a map could be made, which would scale so closely

* Continued from February, 1914, *Proceedings*.

† Author's closure.

Mr. Follett. under all conditions of weather. He has found that map paper swells when the air is moist and that the swelling crosswise of the sheet may be twice that lengthwise, so that the map is distorted and measurements of long distances cannot be checked to the limit mentioned of "$\frac{1}{80}$ of an inch".

The rod used permitted of accurate readings, under good atmospheric conditions. The work was not done in a slipshod manner, as might be inferred from Mr. Brown's remarks. Certain standards, which were considered proper for obtaining data for a map of 1 in 5 000, were adopted and enforced.

The writer regrets that he did not note the table on page 235 of General Barlow's report, and reproduced by Mr. Smith, as Table 25. He took the distances between monuments from the official report, signed by all six commissioners—three on each side—supposing them to be correct. It is gratifying to know that his chaining was so good.

Two of the Brandis transits used on this Barlow work fell to the writer, and he used them on an extensive survey of the lower Rio Grande in 1897-98. They were a source of constant worry, as the stadia wires would vary slightly in their intercept and in neither was the interval 1 to 100, nor was the middle wire half way between the other two. Frequent interval determinations were necessary.

The use of the needle, as suggested by Mr. Smith, does not appeal to the writer, except for very rough preliminary surveys. He does not believe that any man can read a needle closer than to 5′ of arc, and his error may be 10′, and an error will be introduced which is many times that which arises from the neglect of $f + c$. The diurnal variations will also affect the needle readings. This may do for very rough work, but not where a fair degree of accuracy is desired.

It was not the writer's intention to intimate that he sent instruments to the makers for adjustments other than that of the stadia wires. In fact, he has never sent an instrument to the maker for any adjustment. If the stadia wires did not subtend 1 in 100, he used a reduction table. He merely suggested the expediency of having wires reset.

The point in the paper most criticized was the ignoring of $f + c$ and the absence of interval determinations. The writer should have stated that the intervals of the transits used on the stadia work were tested at the beginning and at the close of the survey, and that one, with which the instrumentman did not get as good closures as the other men, was tested two or three times during the progress of the work.

Assuming that $f + c$ is to be ignored in the field work, the writer would enquire how a man is to proceed to make a reduction table when, the hubs having been accurately set by steel tape 50 m. apart and the transit set up $f + c$ (which was, for the transits used, if the writer's

memory serves him, about 0.3 m.) back of the first hub, the instrument-man states that he reads exactly 50 cm. on the 50-m. hub, exactly 1.00 m. on the 100-m. hub, etc., and when the other men present, including the writer, check these readings? Three of the five transits used on the survey gave this result. It was known that the intervals of the other two were not 1 in 100 but, as they were not to be used on stadia work, their intervals were not determined. Mr. Follett.

As for the ignoring of $f + c$, the results given in the paper, in the writer's opinion, are sufficient to show its expediency.

The writer agrees with Mr. Brown that the quoting of cost data on any work, stadia or other, is a dangerous proceeding unless all the conditions are fully stated, and that even then it may be misleading. It was the intention to compare the cost of this with similar work. He cannot, however, imagine any case where he would feel justified in spending from $1 000 to $2 000 per sq. mile, or even the first amount, on a stadia survey. If property is so valuable that such an expense in making a survey of it is justified, then some method more accurate than stadia should be used.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

GROUTED CUT-OFF FOR THE ESTACADA DAM.

Discussion.*

By Messrs. Frederic V. Abbot, J. F. Ramsbotham, and Harrison Souder.

Frederic V. Abbot, M. Am. Soc. C. E. (by letter).—Mr. Rippey's discussion† is exceedingly interesting, because it carries to much further length a method adopted by the writer in 1899, in constructing a cut-off wall of moderate depth at Lock and Dam No. 2, Mississippi River, between St. Paul and Minneapolis. At this locality the material forming the bed of the river was a very hard combination of clay, sand, and limestone fragments, mixed with a few granite boulders, overlying the well-known, friable sand rock of this part of the Mississippi Valley. Mr. Abbot.

Fig. 19, shows the nature of this material, which underlaid the lock floor and the lock walls. A description of the difficulties encountered in handling this material, which is quickly eroded by rapidly flowing water, and of the methods adopted to overcome them, is given in the following extract quoted from the writer's Annual Report to the Chief of Engineers, U. S. Army, dated July 17th, 1900:

"The seams were all originally water-bearing, and only became dry after long pumping had drained out all the water tributary to them. At a lower level these seams run out into the bed of the river, and in this way passages for water are opened between the river bed and the interior of the cofferdam, many feet below the original bottom of the river at the point where the cofferdam stands. It had been realized from the start that such seams were to be encountered, and to avoid opening up too great a flow at any one time a novel form of cut-off wall was adopted after prolonged study. A slot in the sand rock is made, 2 inches wide and 12 feet long, parallel to the river wall and lying between two longitudinal foundation timbers, by jetting

* Continued from April, 1914, *Proceedings*.

† *Proceedings*, Am. Soc. C. E., March, 1914, p. 779.

Mr. Abbot. a series of intersecting vertical holes 10 feet deep and breaking off any projecting edges between the holes. Most of the eroded material is brought up out of the slot by the jet, and the rest is easily pumped out with the ordinary Edson diaphragm pump and two men. In the usual case this leaves a perfectly clean slot with a considerable flow of water pouring out of it, the water coming from the underground seams which connect with the river. Into this slot a diaphragm made of tongued and grooved three-quarter-inch boards, in two layers breaking joints, is lowered by hand. Three vertical 1½-inch iron pipes, extending down into the slot only a few inches, are inserted, one at each end and one at the middle of the slotted section. A steam pump connected to one of these pipes is now started just fast enough to take care of the flow from the slot without letting it run over, or draining it below the level of the surface more than a few inches, and concrete is rammed into the space above the slot and between the foundation timbers above referred to, which have been firmly drift-bolted down to the sand rock before the slot was cut. As soon as the concrete has set—that is, in about twenty-four hours—the pump is stopped, and water at once flows out of all three pipes. Additional lengths of pipe are screwed on the three till the flow is stopped—that is, till the tops of the pipes are at a higher level than the head due to the pressure in the seams. Thick grout is now poured into smaller pipes passing through the first pipes and reaching to the bottom of the slot. As the latter fills with grout the water is forced back into the seams. The small pipes choke after the grout rises a couple of feet above their lower ends, and they are then raised till they again flow freely. When the slot is full they are removed and the larger pipes are kept filled to the top with the grout as long as any more can be poured in. After about twenty-four hours all the grout has hardened, the pipes are removed, cleaned out, and used over again in another section of cut-off. (Photographs Nos. 7, 8, and 9* show an experimental section of such a cut-off wall which was put in at a high level in the lock pit and then carefully excavated.) The perfect grout filling of the most minute

Note on Fig. 19.—The extreme friability of the material is shown. The method of supporting the timbers on this unreliable material is shown at the top of the picture. The timber is wedged up on wooden blocking so as to leave a space of about 2 in. between it and the surface of the sand rock. The anchor-bolts are then driven and the wedges removed, leaving the timber supported on the anchor-bolts. The surface of the sand rock is then thoroughly washed with water from a hose, all loose matter is removed, and the space between the fresh sand rock surface and the timber is filled with good concrete rammed in place with heavy sledges and steel followers.

Note on Fig. 20.—The two large timbers between which the concrete is seen represent foundation sills. They are fastened to the sand rock every 4 ft. by the 1¼-in. drift-bolts, of which one is seen extending down into the wooden pin, a part of which has been removed. These bolts extend 3½ ft. into the wooden pins, and require a force of more than 9 tons to withdraw them. The wooden pin to the left is shown encased with grout, as it was when the sand rock was carefully removed from around it. The narrow white column about half way between the two pins is the end of the cut-off wall as it appeared when the first 2 ft. of the sand rock was removed. The white marks, looking like splashes of whitewash, are really thin filaments of grout which had penetrated the seams in the sand rock, showing that the cut-off wall is not only a thin impervious vertical curtain in the sand rock, but that the latter itself is solidified for at least 2 ft. on each side. The board in a horizontal position under the concrete is used to prevent the fresh concrete from falling down in the slot alongside of the board curtain, and thus preventing the grout from having a thorough subsequent circulation.

* Reproduced herein as Figs. 21, 20, and 22, respectively.

Fig. 19.—Vertical Face of Sand Rock on the Lower Side of the Excavation for the Upper Lock Gate and Culverts.

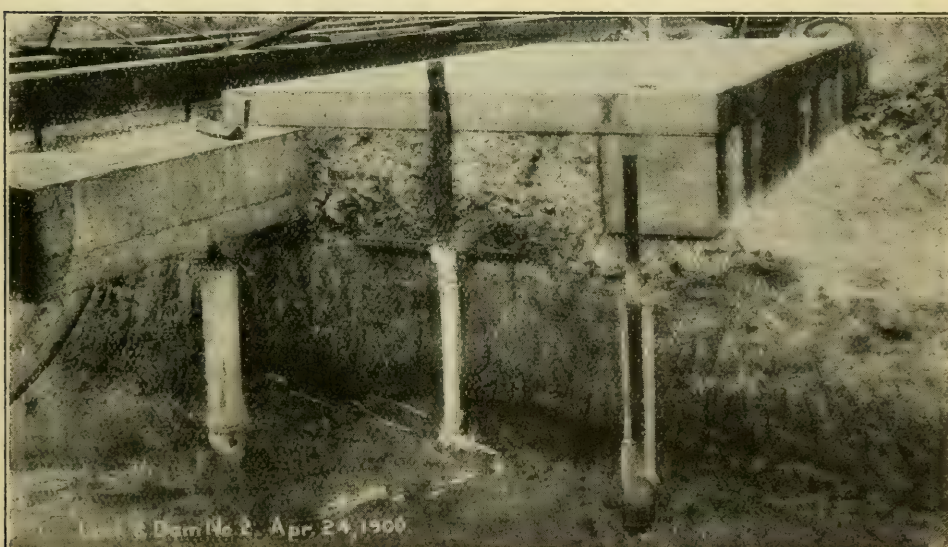


Fig. 20.—Experimental Section of Cut-off Wall, Partly Excavated, to Determine Whether the Grout Filled all Crevices, Etc.

cracks in the sand rock is surprising. An absolutely tight cut-off wall from the bank above the lock to the outer toe of the river wall, and under this toe for its full length and back to the bank below the lock, extending to a depth of 23 feet below dead low water in the river, or deeper in that part under the river wall, is thus secured. Mr. Abbot.

"In some cases no subterranean seams are encountered, and then the wooden diaphragm is inserted and the grout at once poured in around it through pipes leading to the bottom. Experience has shown that in such cases time is a vital element. A slot which does not develop the least flow at first, has in several cases developed very great and troublesome flow in twenty-four hours if not grouted. Most trouble was encountered at the upper end where a very large seam directly connecting with the river had to be cut off. This seam extended nearly horizontally under a space fully 30 feet wide by 80 feet long, and every section of cut-off wall intersected it. Grout poured into one section would pour out of another 30 feet distant. As these underground connections were not suspected, the quantity of water which poured out of three openings into this seam soon exceeded the capacity of the pumps. All arrangements for grouting two of the three sections were completed except the stopping of the flow from the pipes, which in this case were 6-inch in diameter and were running full. It is believed to be unsafe to try to stop suddenly such a flow for fear of blowing up the lock floor. The cofferdam was allowed to fill, thus gradually stopping the flow from the seam, 6-inch risers were screwed into the now submerged overflow pipes and both sections were grouted at the same time from staging erected above the water level. Twenty-one barrels of dry sand cement were required for the grout to fill the slots and seam. In three days the cofferdam was again pumped out and the two springs were found stopped entirely. The third was really outside of the lock walls, being in a section of slot that had been put in to make connection with the cut-off wall under the dam. It was successfully separated from the cut-off slot under the lock wall, which has since been easily grouted."

In the preceding paragraph mention is made of sand-cement. In view of the high freight rates existing at that time between the American Portland cement factories and St. Paul, sand-cement was used in the construction of the land wall of this lock. To insure getting exactly the proportions of sand in the sand-cement, a tube mill was purchased and the cement (Saylor's brand) was ground at the lock site with a very excellent silicious sand found in the vicinity. The concrete made with this sand-cement has stood the extreme temperature conditions at St. Paul, 105° in summer to 40° below zero, Fahr., in winter, for the past 14 years in a surprisingly satisfactory manner. The river wall was built of Saylor's cement, used in the same proportions as the sand-cement in the land wall. A very sharp comparison of the two kinds of concrete has thus been secured. Although the writer has not been able to inspect the work, he has been told by those who have, that the concrete in the land wall is, if anything, superior at the present time to that in the river wall. For

Mr. Abbot. the land wall, the cement was ground with an equal proportion of sand, by volume, and the sand-cement thus produced was used in the same ratio to unground sand and broken stone as if it had been regular Saylor cement.

In connection with the construction of this lock, a great deal of grouting of seams similar to that described was necessary, and in this work great economy resulted from the use of sand-cement in place of undiluted Saylor cement. Experiments were made in order to determine whether sand-cement applied in the shape of grout would harden. A vertical glass tube, 4 or 5 ft. long and about 1 in. in diameter, was plugged at the bottom and filled with a 1 to 1 sand-cement grout and allowed to harden. The tube was broken in 3 days, and it was found that the lower part of the grouting in this pipe was as hard and sound as that at the top. When ordinary sand was mixed in the same proportion with undiluted Saylor's cement, kept thoroughly stirred up, and poured into a similar tube, the sand settled to the bottom very rapidly, leaving a creamy grout with hardly any sand grains in the upper part of the tube. Under these conditions the bottom 2 ft. never set, though the upper 2 or 3 ft. set about as hard as the sand-cement grout did in the other tube. The reason is plain. The specific gravity of cement clinker is not widely different from that of silica. When the silica is in large grains, and the cement clinker is ground fine to make a Portland cement, the two materials separate in water, the larger particles settle quickly and reach the bottom first, and the finer particles, which settle less rapidly, remain near the top of the column. Since making this experiment the writer has always had considerable doubt as to results of grouting with Portland cement stirred up with ordinary sand, but he has no doubt of the effectiveness of grouting with sand-cement ground 1 part of cement to 1 part of silicious sand, provided the grinding is carried far enough to make the sand as fine as ordinary Portland cement. From Mr. Rippey's description, the writer supposes that no sand was incorporated in his grout. If sand-cement had been used, it would seem, therefore, that the cost of grouting might have been somewhat reduced provided the freight rates to the Estacada Dam were high enough to make it pay to install a cement-grinding tube mill.

Note on Fig. 21.—The complete covering of the end of the wooden curtain by the grout is well shown, as well as the way in which the latter filled every irregularity in the end of the original slot. The grout was about 1 week old, and was at the time firm and hard. It was composed of sand-cement ground in the proportion of 1 sand to 1 Saylor's Portland cement. The extreme wetness of the sand rock is well seen in the pond of water at the foot of the section. Though there was no regular flow, the water was oozing from all the pores of the face exposed by the excavation.

Note on Fig. 22.—At the top, the grout covering the end of the diaphragm has been removed to show the boards, and, in the middle of the plane parallel to the diaphragm, the grout has been scraped off to show the flat side of the boards. The rest of this plane is left as it stood when excavated, showing that the grout absolutely holds the sand rock to the boards. The irregular projections at some points in this plane show where there was originally a seam in the sand rock, which has been filled by the grout.

EXPERIMENTAL SECTIONS OF CUT-OFF WALL.

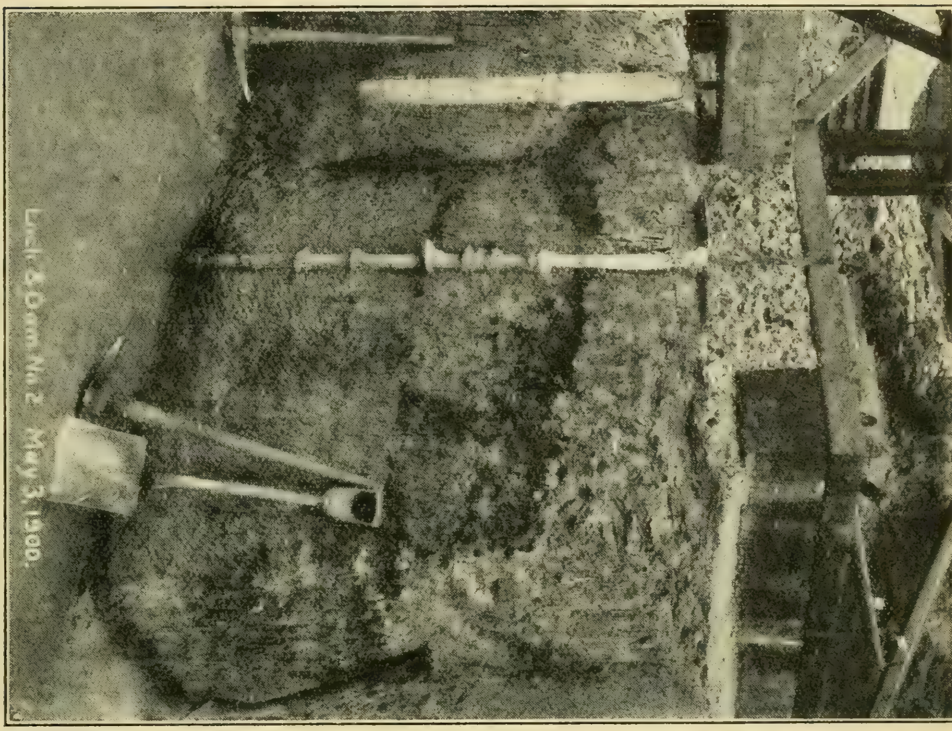


FIG. 21.—WHEN EXCAVATION HAD PROGRESSED TO A DEPTH OF 6 FEET.

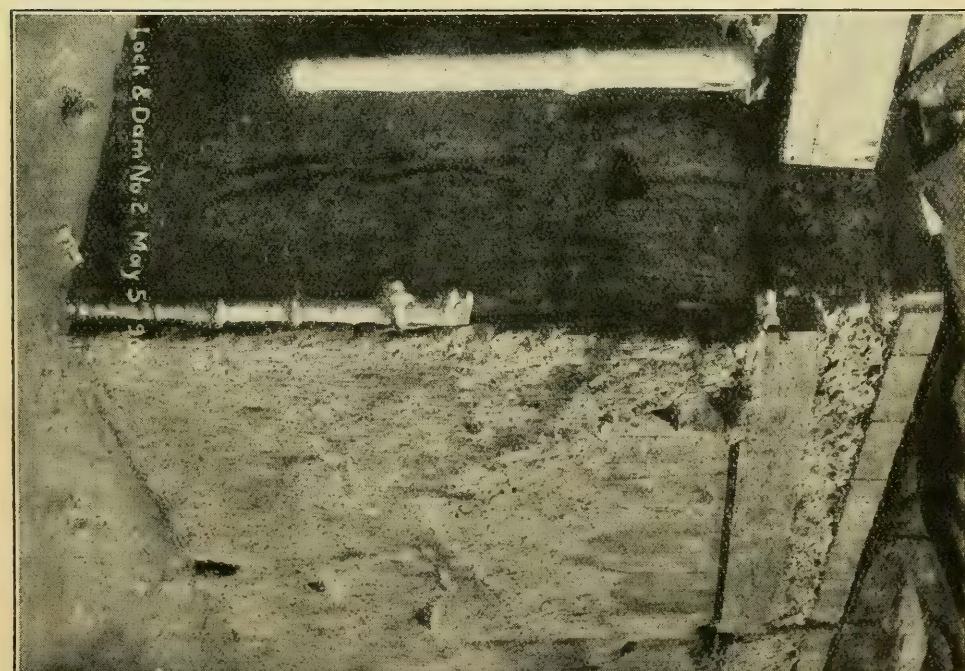


FIG. 22.—A SECTION PARALLEL TO THE DIAPHRAGM OF BOARDS, AND A FEW INCHES, ONLY, FROM IT.

J. F. Ramsbotham, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest and is of the opinion that it will be of value to the entire Profession. He would like to draw some conclusions based on his experience gained on the Fremantle Graving Dock.* Mr. Ramsbotham.

In the first instance he differs with the author in regard to the method of starting with a low pressure of 25 lb. and ending with a pressure of 250 lb. per sq. in. Surely the pressure should have been higher in the initial stages, for, with the depth of hole in use, there was no fear of doing damage, and, at the same time, a better and more pronounced penetration would have been assured.

A pressure of 250 lb. per sq. in., or 16 tons per sq. ft., is certainly very high, and the quality of the rock must have been exceedingly tough to have withstood it. The danger of using such a pressure is considerable, but risk of damage would be lessened by having other holes drilled in the vicinity of the grouting, such holes acting as a safety valve.

The author's summary of the advantages reaped is somewhat disappointing.

From the writer's experience, cement grouting can be relied on to form a cut-off from percolation water, and this experience was gained by witnessing daily a material decrease in pumping.

The holes should be staggered, the front row being 6 ft. from center to center and the back row 3 ft. back from them. The writer considers it preferable to drill the front row first and then check and test the results by the back row, but it is quite impossible to lay down any hard and fast rule. Each case must be judged on its merits.

The whole question of efficiency is problematical, and for that reason, before adopting cement grouting and giving a positive opinion, an actual inspection of the site is desirable.

As regards fortifying bad ground (such as a gravel bed) to withstand a heavy statical load, the writer has no hestitation in saying that by cement grouting this can be done economically and safely, there is no excavation to make and replace with concrete, and no pumping.

Harrison Souder, M. Am. Soc. C. E. (by letter).—It is a fact that very little has been published in regard to the treatment of foundations by the use of cement grout. The interest manifested in the Estacada work causes the writer to regret deeply that his experiences in this line some 13 years ago, were not given to the Profession at the time. However, the identical method of securing a tight cut-off wall—by injecting a cement grout under pressure through drill holes, as described by Mr. Rands—was used by the writer in the construction of the Hinckston Run Dam at Johnstown, Pa., as long ago as 1901. Mr. Souder.

* *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 1942.

Mr. Souder.

The fact that this method of grouting was used in the foundations of this work prior to the Estacada operation is brought out in a brief description of the Hinckston Run Dam published in 1909 by the late James D. Schuyler, M. Am. Soc. C. E.*

At the time this method was adopted at Johnstown, the only information the writer had on the subject was that gained from watching the grouting of the tunnel backing of the deep sewers built in connection with the Pennsylvania Avenue Subway and Tunnel and in closing leaks in the Torresdale Conduits, in Philadelphia; and especially from a report† by F. V. Abbot, M. Am. Soc. C. E., Major (now Col.) Corps of Engineers, U. S. A., Engineer in Charge of Improvements of the Mississippi River at St. Paul and Minneapolis. The report describes an extremely interesting method of forming a tight cut-off wall by using cement grout, exactly after the method suggested by Mr. Rippey on page 779‡ and illustrated by Fig. 16, except that the diaphragm was made of wood and the grout was applied under hydrostatic pressure sufficient to overcome the head of the water flowing through the rock seams.§

As the work at Johnstown was in the seamy shales of the soft-coal formation, a detailed description of the cut-off wall may be of value even at this late day. To add to the clearness, and as a matter of record, a brief description of the dam is given.

The Hinckston Run Dam.

In 1900 the Manufacturers Water Company, after a comprehensive study of the problem of increasing the water supply of Johnstown, Pa., and the rapidly expanding plant of the Cambria Steel Company, determined on the construction of several large reservoirs and pipe lines.

In view of the frightful calamity of 1888, caused by the breaking of the South Fork Dam, there was naturally much opposition to the construction of any new dams above the city. Under the direction of Mr. John Birkinbine, as Consulting Engineer, the first of the proposed reservoirs was located 5 miles from Johnstown, on Hinckston Run, a stream which flows into the Conemaugh a short distance below the famous Stone Railroad Bridge. All the streams in this district are essentially mountain streams. The valleys are narrow and the hillsides steep and rocky, and therefore the run-off is rapid. The average yearly rainfall is 36 in., but it is not well distributed, the major part coming down in the winter and spring; though some extremely heavy and sudden downpours occur in the summer.

* "Reservoirs for Irrigation, Water Power and Domestic Water Supply," p. 518.

† An abstract of this report was published in *Cement*, January, 1901.

‡ *Proceedings*, Am. Soc. C. E., for March, 1914.

§ The reader is referred to Col. Abbot's discussion, which contains an extract from his report, and several illustrations.

The original Hinckston Run project called for an earth dam, 60 ft. high, to retain some 400 000 000 gal. of water, with a depth of 45 ft. at the breast. Mr. Souder.

The intention, as stated, was to build a dam 60 ft. high, with a clay core, but, as an unlimited quantity of cinder from the steel plant was available, it was decided, after the work was started, to use this as backing for the dam, in place of earth, and eventually to fill the whole valley below with this material, thus rendering the structure practically unbreakable. In view of this and the additional expense incurred in making the cut-off tight, the proposed height of the dam was increased to 80 ft., and later to 85 ft., above the original creek level. This gave a total maximum height above the bottom of the core-wall ditch of 112.8 ft., a depth of water at the breast of 73½ ft., and a capacity of 1 100 000 000 gal. The lake thus formed is 1¼ miles long. The water-shed above the dam is 10.75 sq. miles.

With the consequent great increase in pressure, extra care was needed, and every precaution was taken to make the structure watertight.

The site selected for the dam required a rather long breast, but gave an excellent opportunity to construct a spillway with natural rock bottom, and was close to several large deposits of excellent clay lying within the reservoir site.

Work on this dam was started in the fall of 1900. The writer took charge as Resident Engineer and Superintendent of Construction early in May, 1901. At that time the dam site had been cleared of timber and partly stripped, and a small section of core-wall ditch had been excavated to a depth of 12 ft.

The first work done was to strip the surface of the dam site of all soil and vegetable matter. This was generally 3 ft. deep, but over a large area, near the creek channel, it was necessary to remove material to a depth of 8 or 9 ft. owing to numerous beds of muck, old logs, etc. All this material was deposited in the rear of the embankment proper. The remainder of the reservoir area, some 105 acres, was cleared of all timber and brush, but the surface soil was not removed, as the water to be stored was intended for mill purposes only.

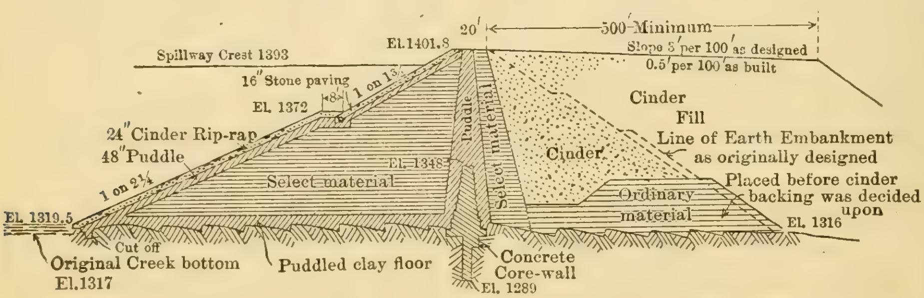
Work on the dam proper was suspended on rainy days and during the winter.

The cross-section of the dam as built is shown by Fig. 23. The lower inner slope is 1 on 2¼, with 4 ft. of puddle and 24 in. of cinder rip-rap. The slope above the berm is 1 on 1¾ with puddle lining diminishing to 2 ft. thick at the top. The facing is hand-laid stone paving. The puddle wall is 16 ft. thick at the top of the concrete core-wall, and diminishes to 4 ft. at the top of the dam.

Early in July, 1901, when the core-wall ditch had been carried down to hard rock at what was thought to be the proper depth, some test

Mr. Souder. holes were bored through the bottom to determine the character of the rock below. This disclosed a layer of hard sandstone a few feet down, with considerable water flowing below and above it. It was decided to deepen the ditch considerably, in order to get below any rock strata that might come to the surface within the flooded area, and to substitute a concrete core-wall for the clay one originally proposed.

An air compressor plant was installed. This was a 14 by 18-in. Ingersoll-Sergeant machine capable of driving two rock drills and six pneumatic rammers. These rammers were used in tamping concrete and also in puddling clay in such places as could not be covered by a 10-ton steam roller which was also supplied at this time in place of the 3-ton horse roller in use.



MAXIMUM CROSS-SECTION, HINCKSTON RUN DAM

SCALE OF FEET

0 10 20 30 40 50 100 150

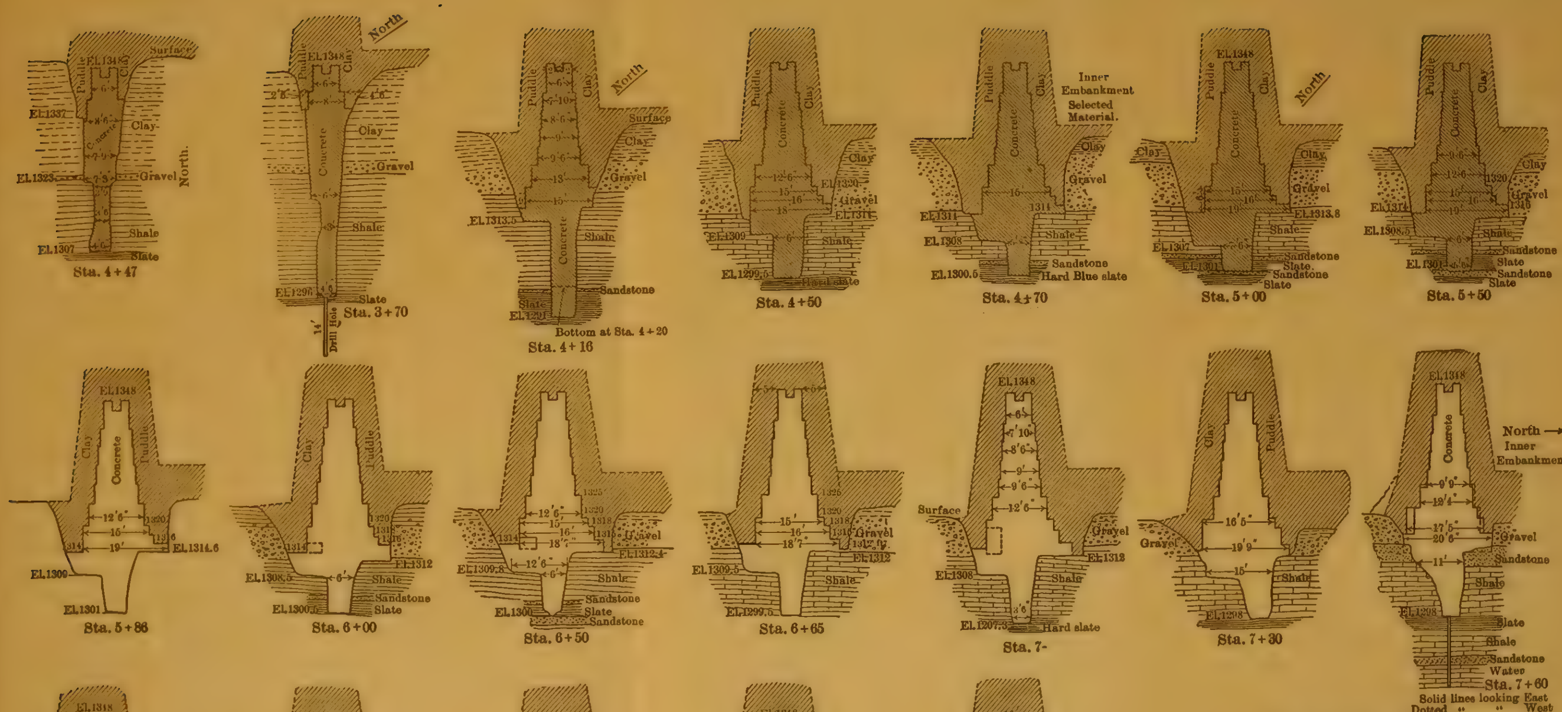
Clay puddle and select material spread in 6" to 10" layers and rolled with 10-ton Corrugated steam roller.

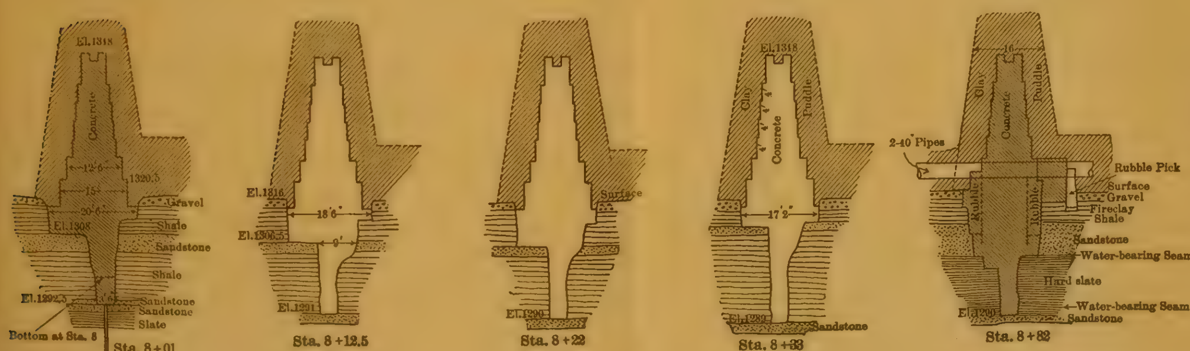
Fig. 23.

The finished ditch averaged in depth 25 ft. below the grubbed surface in the valley, but reached 50 ft. at the ends. The shale was excavated with picks, but the harder rock was loosened with light charges of dynamite, care being taken not to shatter the foundation or open up the seams. Plate XII shows typical sections through the core-wall and ditch, the variable character of the strata cut through being indicated. The advisability of cutting off the underflow to as great a depth as possible was realized, and it was determined to remove the shale down to the sandstone and try to cut off the flow below by forcing in cement grout under air pressure.

Drill holes, 2 in. in diameter and from 10 to 16 ft. deep, were drilled through the rock, averaging about one hole per linear foot across the valley. Iron pipes, 2 in. in diameter, 18 in. long, and threaded on one end, were cemented into these holes. Portland cement grout was poured into them and then air at a pressure of from 30 to 60 lb. was applied.

The first holes were approximately 6 ft. apart. They were drilled generally 10 ft. below bed-rock. The first hole drilled was marked No. 1 W., the second No. 2 W., 5 ft. distant, the third No. 4 W., 17 ft. distant.





Note- Drill holes 2" diam. in bottom of ditch; 10 to 16 ft. deep between Stations 8+60 and 8+00; and 6 to 10 ft. deep between Stations 8 and 8+70. Cement grout forced into these holes under 60 lb. air pressure.

CROSS-SECTIONS
CORE-WALL AND DITCH
HINCKSTON RUN DAM.

Mr. Souder.

After No. 1 was grouted, a small pit, or sump, was excavated through bed-rock and into the shale below. Signs of grouting were visible 2 ft. below the sandstone, which was encouraging. After some twelve holes had been drilled within a distance of 66 ft., an effort was made to determine the location and extent of the water seams below bed-rock by using a solution of permanganate of potash. In a letter to Mr. Birkinbine, dated July 22d, 1901, the writer described this test as follows:

"In the main core-wall ditch I made some tests to determine the location and length of seams under bed-rock. Mr. Hyde, the Cambria Chemist, brought some potassium permanganate out. We put several buckets full of the solution into the receiver and connected it with hole No. 3 West and then turned on the air. The color showed in the sump and in holes Nos. 4 and 5 and slightly in No. 6, entering the last hole at least 2 in. from the surface. This showed that 2 in. and 3 in. of shale had not been stripped from bed-rock, also that the seam extended from No. 3 hole beneath bed-rock to the sump. This test was not altogether satisfactory, so we put a pipe in No. 6 W. hole, from which a strong clear stream of water flows. Put some permanganate in the tube and connected the air hose directly to the tube. The color showed in Nos. 4 and 5, in fact, as far as the sump. About this time the pump valve got out of order and the water rose in the trench, covering most of the holes. Then I put on full air pressure, and after a time it began to appear all along the trench bottom from the sump to hole No. 12. Up to No. 10 it was very distinct, at Nos. 11 and 12 the effect was not quite positive. Keeping the pressure on, I had the sump drained, and found the air blowing in at the bottom and on the far side and lower side.

"Evidently there are seams underlying the length of bed-rock. The small y's on the following diagram [Fig. 24] indicate where the air blew out."

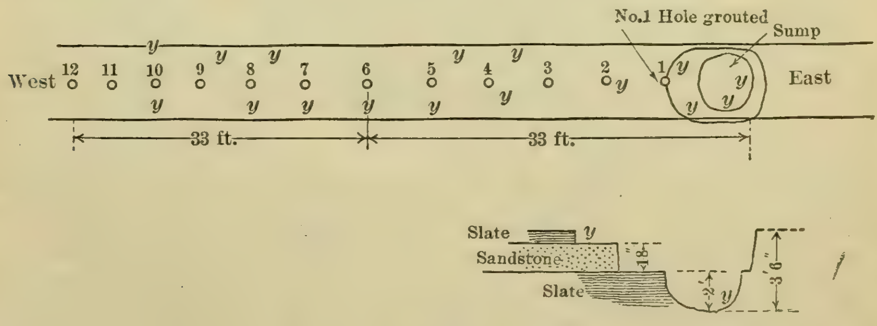


FIG. 24.

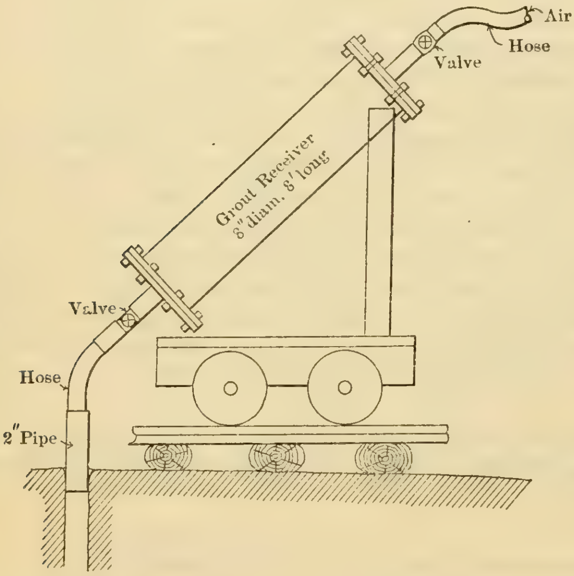
Fig. 25 is a sketch of the first contrivance or receiver devised for applying the grout. It was a cylinder, 8 in. in diameter and 6 ft. long. A screw flange was provided at top and bottom, and a steel head-plate was bolted to each end, with rubber gasket packing. The top bolt holes were open to allow quick removal of the lid. A 2-in. pipe with plug cocks was provided at the top and bottom. With a short hose, the cylinder was coupled to the pipes in the holes. The

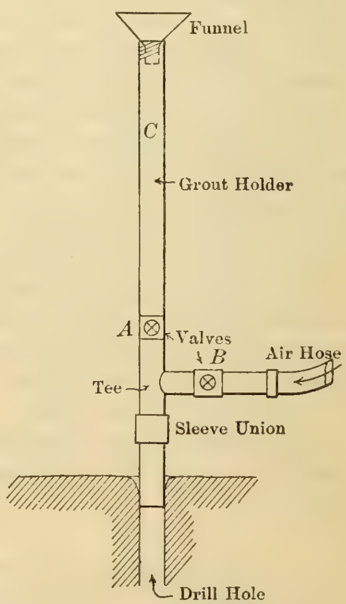
Mr. Souder.

cylinder was filled with grout; the valve was opened; the grout ran into the drill holes, and air pressure was then applied at the top. The contrivance was mounted on a truck running on a track in the bottom of the ditch. After trial it proved to be too slow and cumbersome, and another method was devised and operated satisfactorily. Fig. 26 is a sketch of this final arrangement, and Fig. 27 shows it in operation.

The method of grouting was as follows:

A 1-in. pipe, long enough to reach to the bottom of the hole, was inserted and air was applied to blow out water and dirt. Then a tee and the pipe, *C*, were attached to the tube in the drill hole with a sleeve union, as shown. The cock, *A*, was closed, the cock, *B*, was opened, and air was applied, thus forcing the water out of the hole and into the





DEVICES FOR GROUTING

FIG. 25.

FIG. 26.

crevices and near-by drill holes; meanwhile, the pipe, *C*, was filled with grout, and the air hose was connected at the top. Then *B* was closed, *A* was opened, and air was gradually applied at the top of *C*, forcing the grout down into the crevices. The pipe was refilled about every 10 min. until the hole would take no more. The apparatus was then removed and a cap was screwed on the tube in the grouted hole. After a given length of ditch was grouted in this way, and sufficient time had elapsed to allow the grout to set, test holes were bored within the grouted area and the process was continued until there was no indication of water flow below the bottom. The greater part of the bottom was grouted successfully in this way, but, as explained later, the grouting scheme was abandoned where the core-wall ditch entered the side-hills.

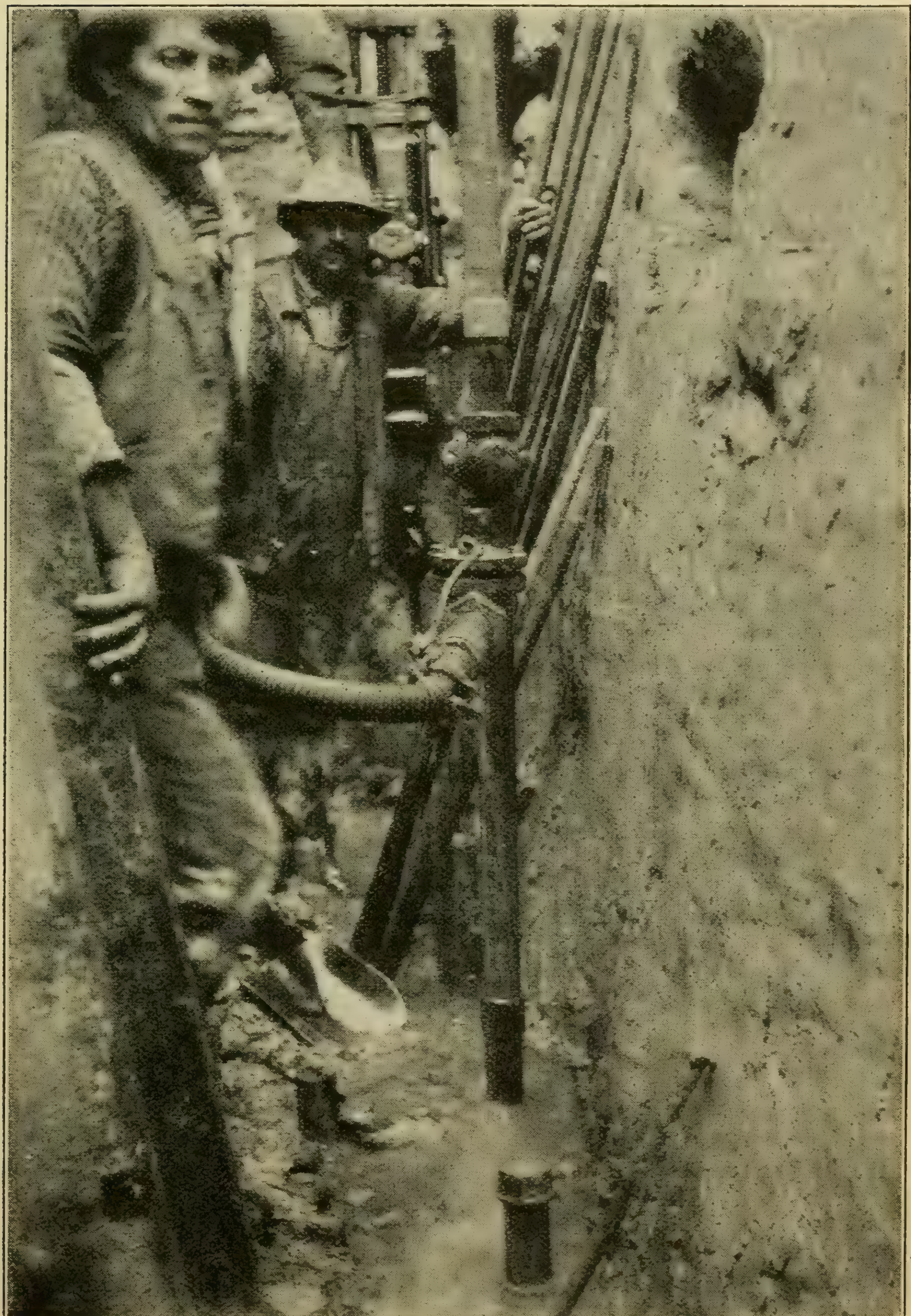


FIG. 27.—METHOD OF GROUTING UNDERLYING SEAMS IN CORE-WALL DITCH. GROUTED HOLE WITH CAP, IN FOREGROUND. CEMENT PLASTER ON SIDE-WALLS OF DITCH.

Descriptive of this work, the following extracts from reports by the writer, made at the time, may be of interest: Mr. Souder.

EXTRACTS FROM REPORTS ON GROUTING TESTS.

July 12th, 1901.—"Finished drilling second drill hole. Strip rock near first hole. No signs grout at 3 ft. distance."

July 13th, 1901.—"Excavated to hard rock and then drilled holes every 6 ft., 10 ft. deep. (Later, 16 ft. deep.)

"Cut away 18-in. layer of rock to within 2 ft. of first drill hole grouted and discovered evidences of grouting. Two of the new drill holes flowed water, which on test comes mainly from within 3 ft. of surface of rock."

July 17th, 1901.—"Test of setting of grout; after 2 hours the neat cement grout hardly set enough to resist anything like a current of water. Addition of sand helps it. Recommend quicker-setting cement."

August 3d, 1901.—"There is a thin layer of shale and slate on the sandstone at Hole No. 11 E. becoming thicker to the east at Hole No. 20; the sandstone is 6 ft. below the shale. This explains why water is found at two levels. On No. 19 E. we had water at 4 ft. and again at 15 ft. To-day grouted several holes. The first was No. 13 W. From this hole a fairly strong stream was flowing out of tube 12-in. high. I first forced air in the hole without cement. Air bubbled out of Hole No. 11 W. very weakly but quite strongly from Nos. 12 W., 14 W., 15 W., and 19 W. Made a smooth-flowing grout of 2 bags cement, 1 bag very fine screened sand, and $3\frac{1}{2}$ buckets water. After the air had been on 30 min. a strong cement color appeared in Hole No. 15 and weaker color in Hole No. 14. Kept pressure on for 1 hour. The grout appeared rather soft in the hole, and after a time a very small thread of water came through it. Flow of water from No. 12 did not diminish appreciably, but carried slight cement color. Difference in level of grout in cylinder, before and after, $5\frac{1}{2}$ in. Next grouted No. 11 W., forcing in $3\frac{1}{4}$ in. of grout after $\frac{3}{4}$ hour pressure, there was no perceptible difference in flow from adjoining holes. There was no water flowing from No. 11 before grouting. Hole No. 8 took $2\frac{1}{2}$ in. of grout after $\frac{3}{4}$ hour. No water flowing from the hole before grouting."

August 6th, 1901.—"In the core-wall ditch we have now 13 holes grouted. To-morrow I will move the drill back and put in some intermediate holes to test the grouting. I must arrange in meantime to drain the east end of the core-wall ditch in order to uncover the sandstone, which dips rapidly into the hill. The shale, being more than 6 ft. thick, is giving considerable trouble to the drillers, so think it advisable to suspend drilling in the east end of the core-wall ditch until the sandstone is uncovered. The following is a description of grouting of holes mentioned in letter of last evening:

"Hole No. 4 W.: water standing in tube about 1 ft. high. Cylinder with grout placed and air on for 1 hour. Valve on tube closed for $\frac{3}{4}$ hour longer. Hole contained 2 ft. cement in bottom, with water above. Rammed thick mortar into tube and put on pressure again for $\frac{1}{2}$ hour, $8\frac{1}{2}$ in. of grout went into the hole.

Mr. Souder. "No. 2 W.: no water flowing from this hole. Connected air hose directly to the tube, forced the water out of the crevices. Then poured in two buckets of grout (1 cement, 2 sand) after having air pressure for ½ hour, the grout had settled in the hole 2 in.

"No. 12 W.: strong stream water flowing from this hole at beginning. Forced water out, the air coming out at the sump near hole No. 1 W. Put in one bucket sand and cement dry and put on air for 10 min., after which time the hole was empty. Then put in second bucket dry grout and put on air. This stopped air from coming out at the sump. The hole was practically empty. Then put in a bucket (3 gal.) of wet grout and put on air for 15 min., which left the hole plugged. Hole No. 7: no water from this hole at beginning. Pumped water out by hand-pumps. Put in a bucket wet grout, and after pressure on 15 min. the grout had settled 2½ in."

Again, on September 9th, the writer stated:

"The core-wall ditch, where the pipe line crosses, is coming into a very hard and tough stone, massive, with little evidence so far of stratification or cleavage planes, and but little water.

"In the main core-wall ditch appearances are not so pleasing, I have had four test holes driven.

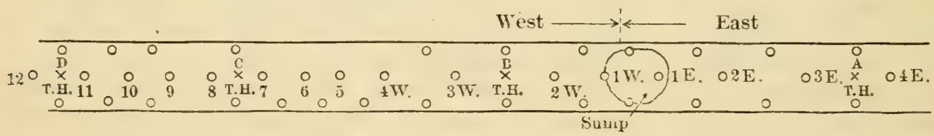


FIG. 28.

They are marked $\underset{\text{T. H.}}{\overset{\text{A}}{\times}}$ [on Fig. 28]. There was a considerable flow of water from all of them; least from B, and the strongest from C and D. Water was struck in all these holes between 11 and 16 ft. Putting air on at A, it came up at the sump, also through crevices in the bottom of the core-wall ditch near A, also in test holes, B and C. Test hole, D, was not finished at the time. Hole A took 8 buckets of grout; Hole 3 took 3 buckets.

"When air was on C air came out rather strongly at the sump, and at test hole, D, and from bottom of core-wall ditch between holes, T, and 6 W., 7 and 8 W., and 8 and 9 W. After putting in 4 buckets of grout and allowing to stand some time, cement got hard in the valve. Meanwhile we grouted test hole, D, took valve off C and found grout quite wet in C, and began to be forced out by air from D. Put long tube on C and filled with grout and applied air again on C. Then went back to D and fed it more grout, giving it in all 8¼ buckets. Air ceased to bubble up elsewhere.

"It would appear from these tests that we have a fairly tight bottom down to 10 ft. depth, the depth to which the middle holes were drilled. There is, no doubt, however, a considerable flow of water at 10 to 16 ft. depth.

"To expedite the work of drilling in part, and mainly because I think we will get better results in this work, I abandoned the old method of diamond spacing for the bore holes and started this after-

Mr. Souder.

noon to drill the holes right in the middle of the core-wall ditch and spaced but 2 ft. apart. This will give us the same number of holes for a given length as previously, but, being closer, they are more likely to give us a continuous cut-off wall. At the same time, four holes can be drilled from two settings of the bar, as against three settings required for the old system."

This is shown by the sketch, Fig. 29.

"First position of bar, *A*, second, *B*. First position of drill on bar, *a*, second, *b*. Simply turn clamp over and reverse the drill in the clamp. I think this will be more satisfactory. Let me have your opinion."

To this Mr. Birkinbine replied:

"I am in receipt of your letter of yesterday, and read with interest your memoranda concerning the test holes. Remembering the fact that the first test holes were only 10 ft. deep, and we got more water in the deeper holes, your results would indicate to me that the section covering the shallow holes had probably been grouted about to their bottom, and that what you are encountering now is a practically continuous stratum below this 10 ft.

"I am inclined to believe that the appearance of air in the floor of the core-wall ditch is attributable mainly to crevices in the upper stratum, where the weight of the material is insufficient to resist the pressure. I think your plan of reversing the bar so as to drill two holes from one setting is a good one, but I am inclined to believe you can afford to separate the holes by more than 2 ft. Of course the holes from one setting will be but 2 ft. apart, but I think there could be an interval of 3 or 4 ft. between one pair, and the next pair. As to drilling these holes in the center of the core-wall ditch I should look for better results if, whenever a longitudinal crevice is evident, you drill close to this, *i. e.*, I would put the holes either to one side or the other of the core-wall ditch, according to the location of these fractures."

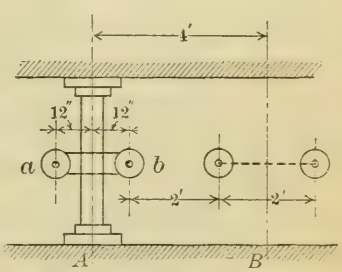


Fig. 29.

Neat cement grout was used after this, the sand being omitted. The proportion was 1 bbl. of cement to 75 gal. of grout.

At the east end three large vertical open crevices were uncovered. (Fig. 30.) The flow from these was so considerable that it was necessary to put in a force pump with a 4-in. suction and a 3-in. discharge, in order to keep the ditch clear. As the grouting method, after continued trials, did not give satisfaction at this end, it was decided to take up the bottom until the principal water strata were reached. This was done, and concrete was placed in the ditch, all the walls being plastered first with a rich cement mortar worked in

Mr. Souder.

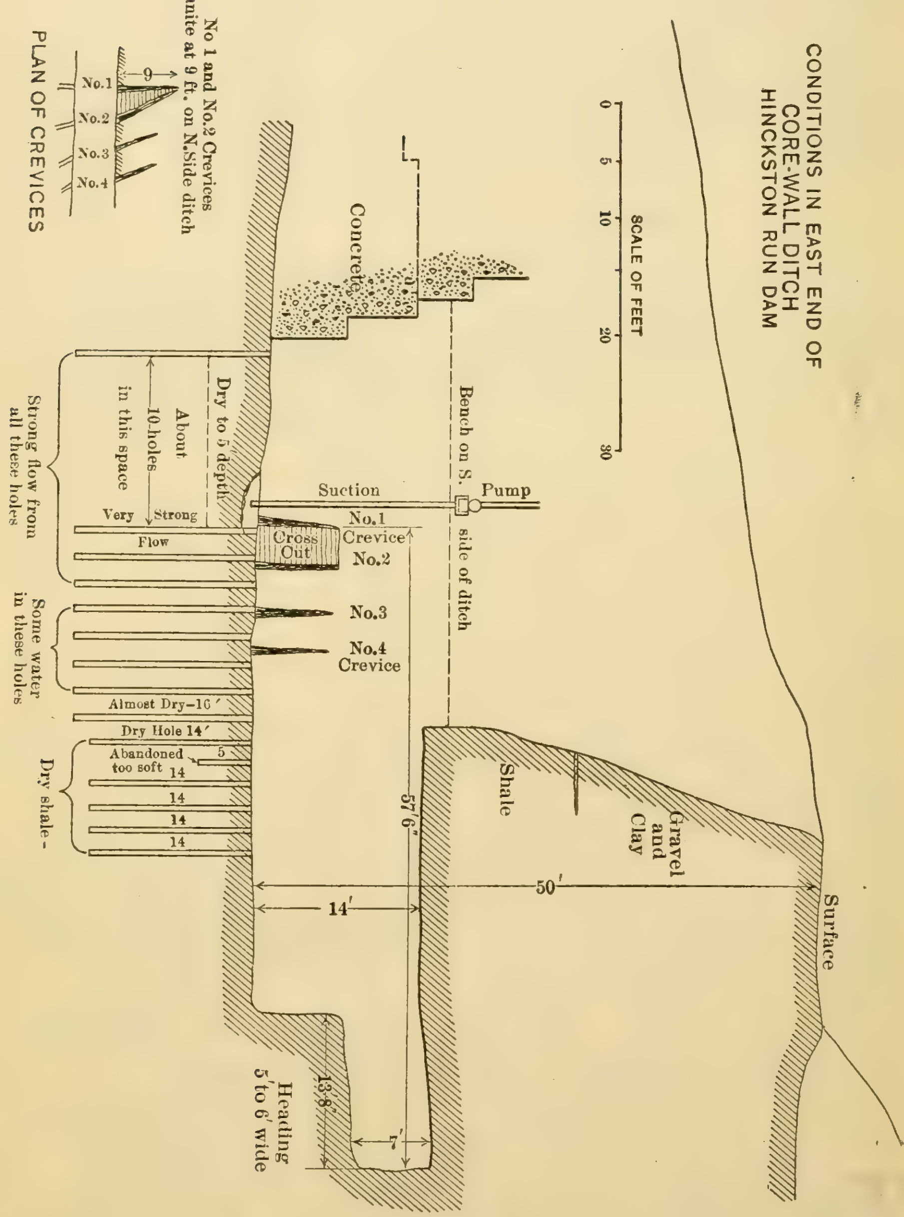
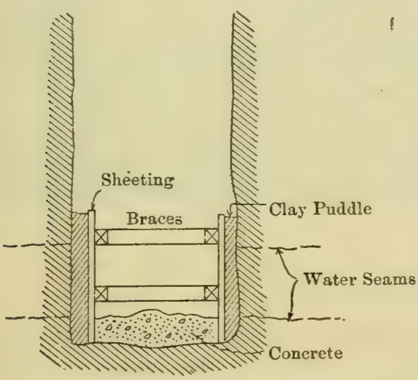
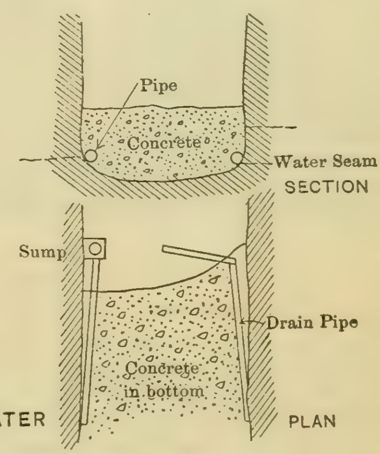


FIG. 30.

with trowels. Two coats of plaster were applied on the north or reservoir side and one on the south side. The suction pipe of the pump was built in concrete, and carried up with the wall. The strong flow of water in this section of the ditch made it difficult to place the concrete for the core-wall without having the cement washed out before it set. Mr. Souder.

At first, the method indicated in the sketch, Fig. 31, was tried, namely, a line of 1-in. sheeting was placed as shown, and clay was rammed between the sheeting and the rock to stop the water flow. Concrete was placed between the sheeting which was raised gradually as the concrete was carried up. This did not prove wholly satisfactory, and the method of piping the water directly to the sump was adopted, as shown in Fig. 32. This proved successful. Where there was too great a flow of water, plastering could not be done, but con-





METHODS OF SHUTTING OFF WATER IN THE CORE-WALL DITCH

FIG. 31. FIG. 32.

siderable neat cement was dumped in along the walls with the concrete and well worked in. After the concrete was well set, the pipes were plugged and the flow of water was shut off. As the concrete was carried up, the water came out of the ditch walls higher up, showing that the underflow had been intercepted. The core-wall ditch was extended well into the hills on each side of the valley, and test holes were bored for water, none being found, because, after a certain distance, the rock became hard and massive and free from seams. At the west end of the ditch, there was as much trouble with water. It would seem that the underflow of the whole valley was concentrated at this point, the rest of the ditch having been grouted and the flow cut off.

The drill-hole grouting method failed in the west side of the valley, as in the east, and here the bottom was also taken out, down to the water strata, and the water fought inch by inch by piping it

Mr. Souder. from the streams to the sump, as before described, and the ditch was completely filled with concrete.

The proportion for the concrete in the core-wall was 1:2:5, generally; but, at the bottom, it was much richer in cement, which was not spared in efforts to make a tight job. Near the crevices a proportion of 1:1:2½ was used. These proportions had to be varied, also, to suit the sizes of the stone supplied, which varied from ½ in. at times to 3 in. The top section of the wall was made of 1:3:6 and 1:3:7 cinder concrete. The concrete was mixed in a machine of the continuous-mixer type consisting of a long square revolving box with a helix at the back. The machine was not wholly satisfactory, but was the best to be had. The concrete was received in ½-cu. yd. dump buckets, carried on small trucks running on light track to the derricks; it was then lowered to the bottom of the ditch and dumped, as it was not thought advisable to drop it from any height. At first a middling dry concrete was required, but, later, a wet concrete was found to give the best results and was finally adopted for the remainder of the work.

After the core-wall reached above the surface the faces received two coats of plaster, one inside against the wood forms and another after the forms were removed. For the upper part of the wall, the plaster coat on the down-stream side was omitted; and, on the up-stream side, a cement wash applied with a brush was substituted for the second coat of plaster. Where sections of the wall joined, bonding grooves were provided. Fig. 33 is a view of the ditch and the concrete core-wall, showing the bonding grooves. Typical sections of the wall as built are shown on Plate XII.

The concrete core-wall contained 10 840 cu. yd., and required 13 166 bbl. of cement, or an average of 1.21 bbl. per cu. yd. The grouting and plastering took 2 078 bbl. of cement, in addition. The exact quantity of cement used in grout alone is not known.

That the work was successful was shown by the breaking out of some small springs on the inner side of the reservoir at or about the elevation of the top of the core-wall.

The comparatively low pressure of 60 lb. was insisted upon, because it was thought that extreme pressures might do harm in opening up the seams unnecessarily, and thus cause blow-outs below the dam, such as have occurred, the writer is informed, in some recent applications of the grouting method.

The result obtained by all the precautions at Hinckston Run was, not only a water-tight and practically indestructible dam, but the people in Johnstown were completely reassured, thus permitting the company to proceed with the construction of other large reservoirs in the same district.

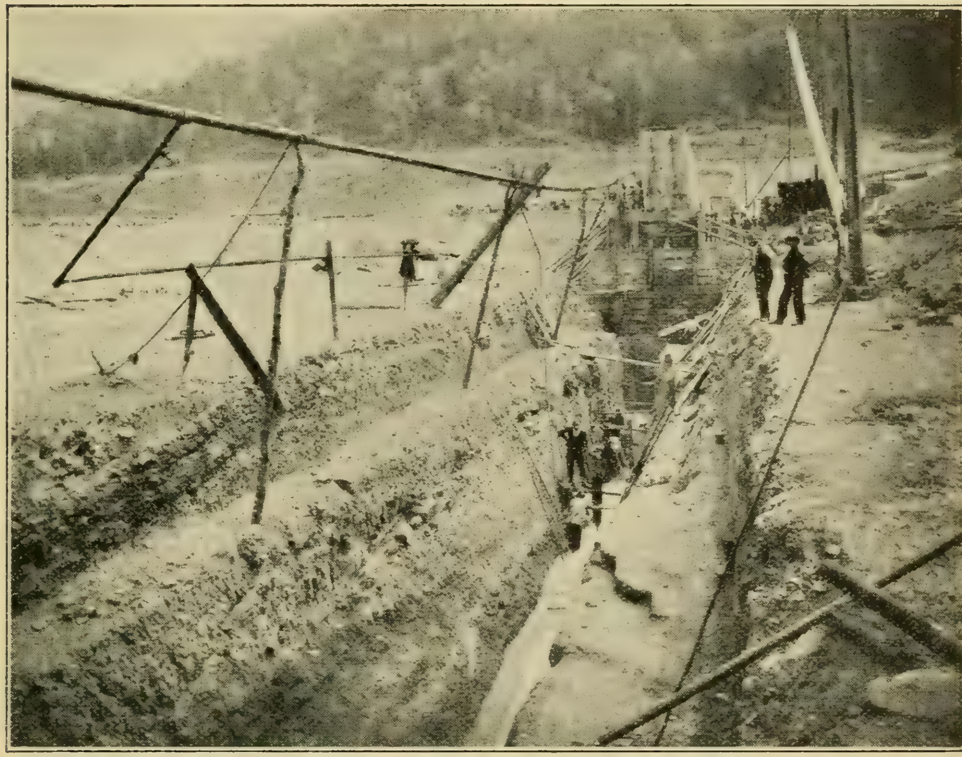


FIG. 33.—WESTERN SECTION OF CORE-WALL DITCH, HINCKSTON RUN DAM. PARTLY COMPLETED CORE-WALL, IN BACKGROUND, WITH VERTICAL BONDING GROOVES.

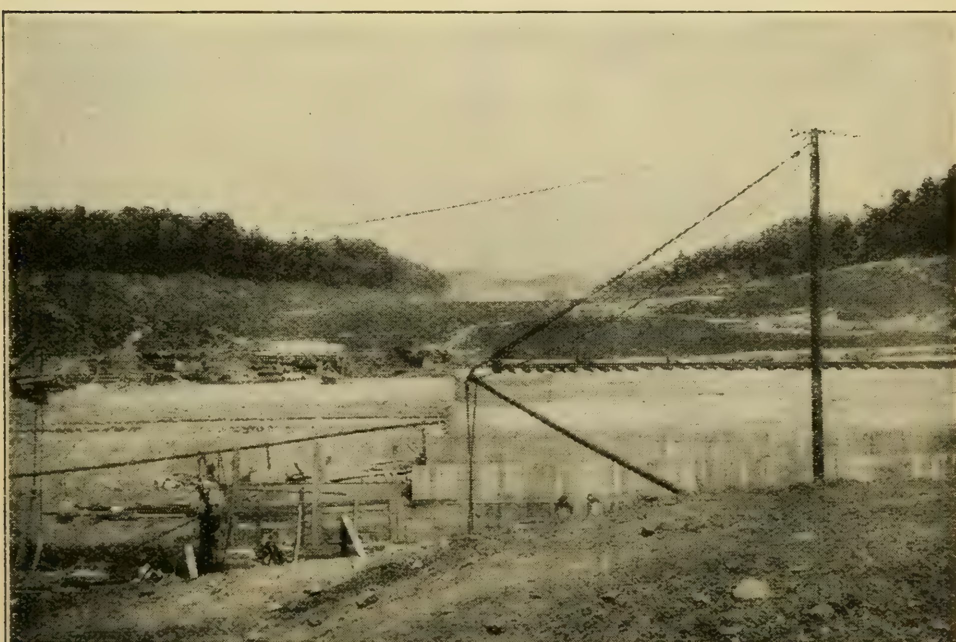


FIG. 34.—PARTLY COMPLETED CORE-WALL, HINCKSTON RUN DAM. LOOKING NORTH, TOWARD RESERVOIR.

The writer considers the grouting for the Hinckston Run Dam to have been successful. In any case, extraordinary care must be taken in applying it, and then only after conscientious study of conditions, and actual tests to determine its availability for the given locality and material. The apparent success of the grouting method for cutting off the underflow at Estacada and other places would indicate careful attention to these requirements. Mr. Souder.

The writer questions the advisability of using air pressure much in excess of 60 lb. in work of this class.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PAINTING STRUCTURAL STEEL: THE PRESENT SITUATION.

Discussion.*

By A. H. Sabin, Assoc. M. Am. Soc. C. E.†

A. H. Sabin, Assoc. M. Am. Soc. C. E. (by letter).—It is not the desire of the writer to engage in unnecessary discussion, but it is desirable to correct misstatements of facts. Mr. Gardner questions the statements made as to the Havre de Grace Bridge tests. It is true that in the paper as first published the percentage of silicates, etc., in Paint No. 6 was too low, but a correction was made at the time of reading; the quantity of red lead was variously estimated at from 65 to 71% by different analysts. Paint No. 10, Mr. Gardner says, contained 10% of litharge, which was one analysis, but the same laboratory reported three other analyses of this paint showing 7.1, 6.8, and 8.5 per cent. Paint No. 11, he says, contained 5% of litharge; the four analyses give 3.51, 3.61, 2.99, and 4.14 per cent. The method used in the Contracts Laboratory‡ at that time gave results too low in red lead; the statement made by the writer was based on private information, and is probably accurate. Mr. Sabin.

Mr. Gardner says:

"The only authoritative paint tests, to determine the efficiency of single pigments as preservers of iron and steel, which have ever been made, are those by the American Society for Testing Materials, at Atlantic City, N. J., in 1908."

If this is true, the writer should have mentioned it in the paper; it seems, therefore, incumbent on him to explain his views as to the matter. In the first place, he objects to the statement that these

* Continued from February, 1914, *Proceedings*.

† Author's closure.

‡ The analyses reported from the Contracts Laboratory may be found in *Proceedings*, Am. Soc. for Testing Materials, Vol. VIII, Plates I to III.

Mr. Sabin. tests were made by the American Society for Testing Materials. The plates were selected and cleaned, and the paints were designed, made, and applied by persons employed for the purpose by an association of manufacturers of mixed paints. The entire scheme and plan was theirs; and the only way in which that Society was mixed up in the matter was by accepting an invitation from the owners of the tests to inspect, by a committee, the prepared plates, as to the history of which they knew nothing except what the owners saw fit to tell them. According to Mr. Gardner, these tests show that no important single-pigment paint, and no simple mixture, such as can be made by an ordinary painter, gave results at all to be compared with the complex paints made by the people who designed the tests. Exception to this general statement may be taken in the case of a few pigments which are too costly to be used, especially the basic chromate of lead, which is described by Mr. Gardner as having "high litharge content", when in fact it contains no litharge at all.

The writer does not regard tests of this sort as authoritative or conclusive. He has before this been on a committee to inspect tests devised by Mr. Gardner, and has been impressed with the unfailing certainty with which they proved Mr. Gardner's contentions; and, for that reason, he can attach no value to them, however unprejudiced and competent may be the inspecting committee. This opinion is strengthened by the observations of A. W. Carpenter, M. Am. Soc. C. E., of the New York Central Railroad, and Mr. S. S. Voorhees of the Bureau of Standards.* Mr. Carpenter observed that in many cases three coats of paint on these panels were going to pieces in 20 months, though similar paints, according to his experience, would be better after an exposure two or three times as long, in New York City. Mr. Voorhees, who had been Chairman of the Committee on Protective Coatings from its beginning, said the Havre de Grace plates, after 5 years' exposure, were in better condition than those of the paint manufacturers' after 1½ years.

Mr. Toch's discussion contains two or three curious misstatements. He says that "every one of these red leads [on the Havre de Grace Bridge] was coated over with a carbon paint". According to the published analyses, as already referred to,† the pigments of Paints Nos. 10 and 11 showed only a trace of carbonaceous matter. He also says that his own paint was No. 12 and was the only one used in only two coats, all the others having three. The tables just referred to show complete analyses of each of three coats of paint on this section, the only one having but two being Section No. 1. It is reported‡ that:

* *Proceedings*, Am. Soc. for Testing Materials, Vol. X, p. 90.
† Also Plate IV, Vol. VIII, *Proceedings*, Am. Soc. for Testing Materials.
‡ *Loc. cit.*, Vol. XIII, p. 338.

"Three paints (Nos. 6, 10 and 11) in Class I, under each of the separate spreading rates, may each well be designated as excellent. * * * What differentiates these paints from all others under observation is the fact that while all the other paints except one furnish their best protection, such as it is, under the 600 sq-ft. rate of spreading and are generally markedly less effective under the thinner film rate, these three show such slight variation under different rates of application as to appear equally protective under either." Mr. Sabin.

This disposes of Mr. Toch's statement that the report "describes as excellent nine paints, three of which are red lead". Mr. Toch also says: "every paint chemist knows that there are some red leads that contain nearly 1% of caustic soda"; as a matter of fact, all red lead is made in a reverberatory furnace, and it is plainly impossible for caustic soda to be present in such a product; if any were there, it would be quickly changed to carbonate; no one has ever seen any.

Mr. A. S. Cushman deplores the fact that the paper "entirely ignores the electrolytic theory of the mechanism of the reactions which lead to metallic corrosion." It may be that this criticism is justified; but the writer's view is that a discussion of the theory more properly belongs in a meeting of chemists rather than engineers. The writer believes that he has read everything Mr. Cushman has written on the subject which he plainly regards as peculiarly his own. The writer's ignorance, therefore, is not due to lack of instruction, doctrine, or reproof, but probably consists in a pernicious tendency to try to check up theory with practice, which, however useful in practical work, is sometimes greatly disapproved by the theorist. What Mr. Cushman regards as the true theory of the corrosion of iron is, in the writer's opinion, that the action of water alone is all that is necessary for such corrosion to go on, and that the accidental presence of anything else is needless and not to be considered. This theory involves the supposition that in all water there is a portion which is ionized; that is, part of the hydrogen exists uncombined with the rest of the molecule, an unstable condition, which results in the formation of new compounds when another substance, as iron, is present. This theory, as regards the corrosion of iron, was first proposed by Mr. W. R. Whitney, now Director of the Chemical Laboratory of the General Electric Company, several years before Mr. Cushman adopted it.* Mr. J. N. Friend, who is Principal of a technical college in England, has been the chief opponent of this theory; he has been supported by grants from the Carnegie Institute, of Washington, and has contributed papers to the *Proceedings* of the Iron and Steel Institute on the subject, in which he believes that he has experimentally disproved the theory in question. A full discussion may be

* *Journal*, Am. Chemical Soc., 1903, pp. 394-406, Whitney; and *Proceedings*, Am. Soc. for Testing Materials, 1907, pp. 211-228, Cushman.

Mr. Sabin. found in Mr. Friend's book.* Mr. Whitney, in his original paper, pointed out the enormously greater theoretical efficiency of carbonic acid, which is probably not denied by any one; and as this acid is a normal ingredient in air and rain, and all natural waters, it has always appeared to the writer that the pure-water theory of corrosion was of doubtful practical value. Further, in view of the fact that no one doubts the existence of electrical disturbances coincident with chemical action in general, and especially as it has long been known that electric instability is provocative of corrosion of iron, it seems to be an arrogant assumption to claim the exclusive application of the term electrolytic to the very limited theory propounded by Mr. Whitney, a restriction never proposed by him.

The fact that iron does not rust in certain solutions was known many years ago. Such facts are explained, some of them, and perhaps all may be, by the modern theories of chemical action; and Mr. Cushman has laudably attempted to apply such knowledge to the paint industry. The question in regard to this is: are his inventions, whether patented or not, of any practical value? The only answer the writer can give is that they do not appear to be. Mr. Cushman seems to believe that a linseed oil film absorbs water from the air, that it is in a sense hygroscopic, and that part of the solid particles of pigment in a dried paint film are really in solution; that this water, which is to all intents in solution in the solidified oil, acts on the iron and causes corrosion, and if some slightly soluble chromate, for example, is in the pigment, it will be in solution in this water and will prevent or forbid, or, as he says, "inhibit" corrosion. It is a very pretty theory, but does it work?

In reply to this, attention is again directed to the official report.† After 5 years' exposure, nineteen test panels, representing as many paints, were removed from the Havre de Grace Bridge, photographed, and then the paint was thoroughly cleaned off and the surface of the metal inspected for rusting. Excepting a few spots, where the coating was obviously injured accidentally, the surface of most of these plates was in exactly as good condition as when first painted. According to Mr. Cushman's theory, all these plates should have rusted under the paint; none of the paints used contained what he regards as efficient inhibitors, and some of them contained large quantities of carbon, apparently lampblack, which Mr. Cushman says is the worst stimulator there is. The paint in all cases was thin, being three coats at a spreading rate of 900 sq. ft. per gal., or a total thickness of 0.005 in., and had been exposed to the moist salt air from Delaware Bay, at the mouth of the Susquehanna River, for 5 years. The writer thinks this proves that a good paint, properly used, keeps out

* "Corrosion of Iron and Steel," 1911, pp. 44-67.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. XI, pp. 175-180.

Mr. Sabin.

air and water; and, as long as it remains intact, it protects the metal; when it has deteriorated so much that it has holes through it, the air and water get to the iron, which begins to rust. As long as there are no holes, no inhibitor is needed; when there are holes, no inhibitor will do any good. No doubt there is a difference in the value of pigments; perhaps a soluble sulphate acts to some extent by conducting water through a film, and perhaps acts on the oil itself; perhaps a substance like carbonate of lime forms a soap with the oil and injures it in that way; but of more practical importance is the greater or less surface attraction between the pigments and the oil; in this respect the differences are great and characteristic. To illustrate this, let us remember that when one end of a small glass tube is put in water the water rises, as we say, by capillary action, in the tube; but, if mercury is substituted for water, the surface about the glass is depressed as though it were repelled. These are cases of surface tension and attraction. In an exactly similar way, some pigments have more or less surface attraction for oil. The most remarkable, and at the same time the most important, case is where, in the manufacture of white lead, the pigment is subjected to prolonged washing with pure water, then allowed to settle, and the wet pulp is agitated in a mixer with raw linseed oil. Although the oil is lighter than water, it displaces the water, mixes with the pigment, and forces out the water, which runs off the top, leaving the lead and oil almost free from moisture. Red lead may be treated in a somewhat similar manner, but of course it is not, as it is a furnace product. As far as the writer knows, no other pigment will act in this way; in most cases the presence of a little water in the pigment greatly obstructs its proper mixing with oil; but pigments differ greatly in this attraction for oil; which may partly account for the widely varying quantities of oil required by different pigments. To give an entirely different illustration: it is common knowledge among all paint makers and users that 1 quart of turpentine will thin a batch of paint as much as 2 quarts of linseed oil, because of the greater fluidity, or less viscosity, of the former. However, if we make a stiff paste with any ordinary pigment, as white lead, we require a certain quantity, say 8% of oil; if we substitute turpentine for half of this oil, it will take, not 2%, as might be thought, but 6 per cent. This remarkable fact leads to the consideration that the paste is a plastic body rather than a viscous one, and the function of the liquid is to stick the solid particles together, which the oil does much better than the turpentine, the surface attraction of which for the lead is very low; but, when more oil has been added, to make paint, which is a viscous but not a plastic body, turpentine is more efficient as a thinner than oil. In a similar way we account in part for the advantage of proper cleaning of steel before painting. It is well known that any ordinary oil easily

Mr. Sabin. wets clean iron and steel, spreads over its surface, and is removed with difficulty; but, if the surface is fouled with anything which does not attract the oil, the latter when brushed out into a very thin film—and paint films are very thin—may break and leave holes, or pores, through which the atmospheric agencies get to the metal.

If, now, the pigment is one which has a great surface attraction for the oil, it is obvious that, when the paint is brushed out to a thin film, the presence of these solid particles in vast numbers will tend to hold the film together, to make it in fact tougher, so that it will be less likely to break into holes; and this is one reason why fineness is one of the most desirable qualities in a pigment; and why such a paint brushes out easily into a thin film, or, as we say, has good working quality. It is also a reason why a paint of good working quality (if it has no counterbalancing defects) is a good protective paint, for it makes a continuous film. Conversely, if the attraction between the pigment and the oil is slight, the paint is less likely to be satisfactory in any respect. Nor is this advantage likely to disappear when the film hardens; for the particles which attract the oil are likely to be more tightly cemented into the mass and form an element of strength rather than weakness; and if, in addition, the pigment is one which shows very little, or perhaps negative, attraction for water, it is likely to show great resistance to atmospheric action.

Linseed oil is an excellent insulator against the transmission of electric currents; and some pigments are themselves non-conductors, though others are the contrary. As it is generally agreed that electric tension is an effective cause of corrosion, it is desirable that paint for metal should contain pigments which will tend to insulate, rather than to break down the insulation given by the oil itself. This is a very different thing from Mr. Cushman's theory that certain pigments embedded in a cement of dried oil will act, in conjunction with pure water (which, in the writer's opinion, could not possibly get there), as a primary battery bringing the iron into solution. Mr. Cushman has elsewhere described the action of certain pigments as like a poison to iron; if correct, which in some cases at least may be doubted, it is because of their unfavorable relation to the oil, which renders them incapable of making good films, and the remedy should be looked for in this direction, rather than by his favorite prescription of a homeopathic dose of some inhibitor.

It is well known that some paints, including some varnish paints, and especially red lead in oil, adhere to iron and steel much better than others. Probably the mixture of a pigment in the vehicle affects its surface attraction to the metal. When we consider that often as much as one-third of the volume of a paint is composed of these solid particles, and that, in the case of a really fine pigment, there

are from 25 to 50 particles overlying one another in a film 0.002 in. in thickness (this supposes them to be 0.00001 in. in diameter, which is within the truth), it may be believed that the action of such a mixture is very different from that of oil alone. Mr. Sabin.

Mr. Carpenter's discussion is interesting and valuable. In Table 1, showing his red lead experiments, it is notable that the quantity of paint used in the second series was 6.3% more than in the first (owing to the use of more pigment), but the surface covered was almost 30% more; and in three cases out of five the working quality of the paint was better, and equal in the other two. If a mixture of 30 lb. of red lead to 1 gal. of oil will work better and cover 28% more surface than a 24-lb. mixture, the fact is worth knowing, and should lead to considerable corrections in common practice.

The relative fineness in these tests was determined by the weight in grammes per cubic inch. The writer does not regard this as a safe method. Certainly, a very fine red lead, after being packed in a barrel in the usual way and then sifted to make it perfectly loose and open, will weigh several more grammes to the cubic inch than it did before being barreled; and it is probable that the volume depends as much on the friction between the particles as on anything, and this, again, may depend on their electrical condition, shape of particles (red lead particles differ in shape according to the material from which they are made, some of which is crystalline), and probably many other unknown causes. At all events, in the laboratory with which the writer is connected, the best-known volume apparatus gives results so widely and irregularly at variance with those of a more nearly absolute method which is checked by actual microscopic measurement, that it is not regarded by the writer as entitled to much consideration. It is possible that a fine red lead may contain considerable litharge, if it is taken out of the oxidizing furnace too soon; also, it is, at least theoretically, possible to oxidize highly a somewhat coarse material; but, in general, the finer the material the higher is the oxidation.

Mr. Carpenter's criticism of lack of definiteness as to the composition of the Havre de Grace tests has already been met by the figures given; attention is also asked to Mr. Wagner's discussion giving his experience with red lead ranging from 96.48 to 98.63% for 5 years, with satisfactory results. It should not be forgotten that, in the Havre de Grace tests, at least two of the red leads were what may fairly be called high grade, and were furnished by some of the largest and most experienced paint manufacturers in the country; and it is fair to infer that these makers desired to make the best possible record, and therefore that they had evidence, satisfactory to them, that this was the best material to use; as it proved to be.

Mr. Sabin. As to why it should be better, the writer must admit that the reasons given in the paper are partly theoretical; but he does not see why he should be denied adopting a theory any more than other people, particularly as this one has nothing novel or original about it, is generally held by the trade, and has been forced on him by the constant experience of a quarter of a century. Age, however, does not make a theory right, any more than it does a man; but it entitles it to respectful consideration; it has successfully served a useful purpose for a long time. There is a broader ground, however, on which all can meet; for all agree that more than half the battle is in getting the paint applied to a clean surface in a smooth and uniform coating; smooth, because resistance to wear is much increased by the absence of ridges, lumps, and grains in the surface, and uniform, because thick masses of paint are waste, and thin places are weak. As Mr. Coombs puts it, a $5 man with a $1 paint is better than a $1 man with a $5 paint. Now the whole history of the improvement of red lead as a paint has been of this nature: to please the consumer, it has been made continually finer in texture, so as to be free from roughness and disposition to run, and lower in litharge, so as to be less active to oil and therefore more manageable. Tendency to settle is not so much a matter of specific gravity, but is caused by coarseness; no paint settles less than white zinc and white lead, which are very heavy but very fine; and no paint is more refractory in its general behavior than a low-grade, red-lead paint which has begun to get thick and viscid. It accords, then, with general experience, that a red lead which consists entirely of impalpably fine particles and has about the same relation to oil that white lead has, will be easily applied and form a smooth and uniform coat; and that, irrespective of the theoretical questions involved, it will practically be better on the average and last longer. This, it is submitted, is in accord with all experience with painting in general.

One more observation is suggested by Mr. Carpenter's discussion which is that it is difficult to determine spreading capacity in a panel test. The painter instinctively tries to spread all alike, so as to give each paint a fair chance; the more conscientious he is, the more his results may vary from what are actually secured in working under average conditions. It may be doubted whether a test of less than a barrel of paint is of much value in this regard.

In conclusion, the writer would explicitly deny the charge, made by Mr. Cushman and his associates of the paint manufacturers' organization, that he claims to have said the last word in respect to the protection of steel, and that there is no need of further investigation of paint problems. Such a charge will not be taken seriously by any member of the Society. Another assumption, that the paper was designed to create an interest in red lead, is really based on the fact that every one is interested in that substance, and when any one has

anything essentially new to say about it, whatever else he says is likely to be neglected. This very fact, that it is a widely known and valuable material, made it proper to mention, in a paper of this character, any important improvement in its manufacture or use, and the fact that the writer is able to speak authoritatively of its composition and character seemed a good reason for doing so. It is not a perfect paint; there is no paint suited for all uses; but that is no reason why we should not know as much as we can about any of them, particularly those which we use the most. Mr. Sabin.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

DISCUSSION ON CONDITIONS OF EMPLOYMENT OF, AND COMPENSATION OF, CIVIL ENGINERS.*

By Carl H. Fuller, Assoc. M. Am. Soc. C. E.

Carl H. Fuller, Assoc. M. Am. Soc. C. E. (by letter).—The writer has been vitally interested in the report and the chart of the Special Committee which has been investigating the matter of compensation of engineers and in the discussion of the subject following its publication. The work of the Committee is certainly thorough and commendable, and, should its efforts result in improving the standards of the average member of the Profession, it will merit the gratitude of every member of the Society as well as the Profession at large. Mr. Fuller.

However, the writer cannot help but feel that the chart fails to represent actual average conditions, owing to several reasons, one of which is the effect of the maximum compensation line, containing several of what may be called abnormally big incomes. It will be conceded that the *Transactions* of this Society, owing to the extremely technical nature of a large portion of the papers, do not represent the ideas of the majority, who have neither the time nor the qualifications to prepare and discuss the subjects presented, but, represent the opinions and results of the labors of a very small minority, who being favored with ability and opportunity, derive pleasure in devoting their time to research and such work. Thus, in the consideration of this subject, it appears that the discussions represent the academic opinions of those whose successes in business, if platted as an income line on this chart, would rise above the average, rather than the opinions of the majority, whose income line would fall materially below this average.

The object of every professional organization is the betterment of its individual members, and the matter of personal income is bound

* Continued from April, 1914, *Proceedings*.

Mr. Fuller. to have its reflex influence on the purely professional or technical aspects of that society. From the very nature of things, every paper presented before this Society reflects the financial success of the writer, and there are without a doubt a large number of members, who, if it were not for the handicap of family and the necessity of providing for them, would be glad to contribute very materially to the *Transactions* of the Society. Therefore, if the Society cannot give serious consideration to the material, individual condition of its membership, it will fail to represent the ideals of the majority; thus, the matter of personal or professional income becomes as much a matter of concern to the Society collectively as it does individually.

Out of curiosity, the writer platted his own income line for 15 years, superimposed it on the chart, and found that, being a very ragged line, it projected alternately above and sometimes very considerably below the average line on the chart, although a modified line of averages would follow the average on the chart quite closely. Nor does the writer believe that his success is materially different from that of the majority of engineers with whom he is acquainted, and the conclusion is reached from experience and observation that this Profession is one of constant change and vicissitude, as few engineering positions can reasonably be expected to be permanent. In no way can the Profession be compared with any other professions or industries, and the completion of a building, a railroad, or other project, naturally releases one or many engineers from an engagement that may perhaps reflect the greatest credit on their ability. Then again, the public in general has a very vague conception of an engineer's work, which, in a measure, is due to its transitory nature; consequently, many irregular employers of engineering services fail to appreciate the attainments of a man qualified to perform this kind of work and incidentally appraise his services at the valuation in dollars and cents that he himself places on them.

Architects, in their societies, have found it no disgrace to define their compensation, and physicians and those of other professional organizations have scheduled the value of their services in varying degrees; therefore, why should not this Society endeavor to establish a higher standard, which will command the respect of other men. There is no doubt that the compensation for engineers is less than that for men in other lines of industry, and as the public very largely measures a man's success and ability by the amount of compensation he is able to demand for his services, it behooves the Society and the Profession to protect its professional honor by demanding for its members fees which are consistent with the services rendered.

Conservation and scientific management have become quite a fetish in the Engineering Profession, yet, as a whole, it is more extravagant and wasteful of its energies than any other body of intelligent labor

Mr. Fuller.

in the world. If this statement is challenged, one item alone will be considered: The average engineering job (not position) lasts from 3 to 18 months, occasionally longer. In the transition from one job to another the average engineer will lose at least a month, and, if his last project has been finished late in the year, he is likely to lose 3 or 4 months. Without resorting to statistics, the writer thinks it may safely be assumed that the average engineer loses on this account fully one-tenth of his time and income, through no fault of his. If this waste can be eliminated—for waste it is—not only will the average income line on the chart be raised, but a very material difference will be made in the minimum line, for this waste naturally applies to those whose incomes fall in the lower half of the chart, and among whom there are many able men. Is not this waste as worthy an object of conservation for the Society to consider as that of the water-powers and forests?

Some one has remarked that a capable engineer will always have a job, but this statement needs to be examined. Is it the man who is capable from an engineering standpoint, or the man who is a good salesman of his services that gets the job? The writer is inclined to think that it is the latter; for, getting a job requires salesmanship of the highest degree, more so than selling rails or cement. Consider the men who maintain large offices successfully. Without any discredit to them, let it be asked how much work they get because of their engineering ability, and how much they get from salesmanship? The latter often brings the larger fees. The writer knows an engineer, whose name has been connected with monumental work all over the United States, who cannot perform the simplest equations, yet, from his sales ability and knowledge of men, he has a large practice. If the writer should mention his name he would be accused of slander. Therefore the statement that, if a man has ability he will always have a job, is fallacious.

From these thoughts arise two ideas: (*a*) Can the Society as a body wield an influence that will tend to increase the compensation of its members and the Profession at large so that it will be consistent with services rendered? (*b*) Can it do anything to prevent the loss of time of its members? The writer thinks it can, and, furthermore, that it is one of the obligations of its existence, because no other body of men is qualified to do this.

There are many phases to this proposition. It is the biggest problem that has ever been presented to the Society. Is the Society going to table it, after some academic discussion, or is it going to seek a remedy? The writer would suggest that a standing committee be appointed to continue the work that has been started.

As to the conservation of the time lost, the Society can, if it will, afford immediate and effective results by the establishment of

Mr. Fuller.

a secretaryship or bureau, under the direction of a competent engineer who knows the work and the men, the duties of which will be to furnish a clearing house for the services of its members. In this the employing member will receive as much benefit as he who is seeking employment, for the reason that a properly conducted department of this kind will try to place the right man in the right place, instead of "bleeding" an unemployed man who can ill afford it. The writer is of the opinion that every member who has used the employment agency to secure help has been disappointed in the results obtained, because the employment agent is only looking for his commission. In one instance an agent offered to divide his commission if the writer would employ his clients and discharge them again when the fee had been paid.

Some member on whom fortune has smiled, in the way of successful positions, may object that it lowers the dignity of the Society to go into the employment agency business; but, who has a greater right to conserve the time and compensation of its membership than this honorable body. Each and every member belongs to this Society for one of two reasons: either to derive some benefit from the organization or to confer some good on his fellow-members. Therefore, while the Society is caring for the intellectual welfare of its members, why not go a step farther, and provide for their material welfare, thereby permitting them to derive greater intellectual benefit. The writer hopes that there will be further discussion on this subject from the majority of the membership whose income falls below the average line, in order that some light may be obtained on the whys and wherefores.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

STORAGE TO BE PROVIDED IN IMPOUNDING RESERVOIRS FOR MUNICIPAL WATER SUPPLY.

Discussion.*

By Messrs. E. P. Goodrich and James L. Tighe.

E. P. Goodrich, M. Am. Soc. C. E. (by letter).—What Mr. Hazen has done in the design of probability paper is unique, and, with easily obtainable information as to the meaning of lines drawn on it in relation to probability formulas, his new device should be of almost as much value to engineers as the well-known logarithmic paper. Mr. Goodrich.

The writer believes that the work Mr. Hazen has done in applying the theory of probabilities to the data on annual stream flow and required storage, is a material advance in engineering science. Mr. Hazen has shown conclusively, in the writer's opinion, that the methods of computation to be followed in utilizing the theories of probability are applicable to water problems. The next step in the solution must be what is normally called that of inverse probability or the breaking up of the probability curve found from actual data in any case (such as that prepared by Mr. Hazen and shown in Fig. 29) into several possible component probability curves. This work is exactly comparable with the analysis which is now so common with reference to tidal and similar observations, which have been found to follow compound harmonic relations, breaking up this combination curve into the several simple harmonic ones which, when recombined, will produce the actual result observed in Nature. In the case of the harmonic curves, the tracing back of the several simple harmonic functions to their causes has been found relatively simple. It is similarly to be expected that, if Mr. Hazen's combination probability curve can be analyzed into two or more simple normal probability curves, in combination with one or more other simpler curves, the causes operating

* Continued from March, 1914, *Proceedings*.

Mr. Goodrich. to produce the latter can be discovered, and further refinements can be worked out in the engineering of water-works design.

Several explanations of the skew curve, which Mr. Hazen found with regard to run-off data, are possible. One such explanation, which seems to the writer more than possible is that a normal probability variation is superposed upon a condition which would produce a periodic variation of long range. Evidently, if the data happened to have been secured for years which range near the minimum of a periodic fluctuation, the exact conditions, with reference to the data found by Mr. Hazen, would be reproduced. In the paper Mr. Hazen makes reference to such periodic variations in rainfall and run-off, and it is the writer's idea that the skew nature of the curve found by him indicates, primarily, that the data do not cover a large enough range of years to be entirely conclusive in the deductions which can be drawn. In the meantime Mr. Hazen's work will represent the acme to date.

Mr. Tighe. JAMES L. TIGHE, M. AM. SOC. C. E. (by letter).—The ingenious and unique methods of analysis followed in the production of this remarkable paper, and the conclusions and results therein set forth, must be of the greatest interest to all hydraulic engineers.

In the earlier days of providing domestic water supplies from catchment areas, little attention was given to the relation between run-off and storage, with the result, naturally, that deficiencies generally occurred in the drier years. After a time these deficiencies led to the establishment of empirical rules or formulas for storage based on low rainfalls, or, where run-off records were available, to the tentative and laborious methods of tabulation.

It was not until the early Eighties that W. Rippl, Docent at the Royal Technical High School at Gratz (Styria), made public his graphical method for the calculation of storage known as the mass-curve method. As originally given, this method was followed in such calculations until the report on the water supply of New York by John R. Freeman, M. Am. Soc. C. E., was published in 1900. In this report a more convenient mass-curve method was shown, in which a single curve served for all rates of draft, whereas, in the original Rippl method, a new curve had to be made for each rate.

In this mass-curve method and in the other methods for calculating storage, it was generally sought to calculate from the rainfall or stream-flow records of the particular catchment area in question, or, if these were not available, from the rainfall or stream-flow records of some other catchment area applicable, the storage required to maintain certain drafts through the driest period covered by the records, and little if any attention was given to other important phases of the question, outside of evaporation, until the presentation of Mr. Hazen's paper.

Mr. Tighe.

In this exhaustive analysis, some new ideas relative to storage, which were never brought into the question before, have been treated at great length. Of these, the amplification of the dry years and their recurrences, the normal storage diagram, and the coefficient of variation may be mentioned. For the latter, hydraulics is under great obligations to the author, inasmuch as it must be admitted that, up to the present time, the science gave no yardstick whatever by which the variation in stream flow could be determined.

The coefficient of variation proposed is an ingenious one, and will not only serve the purpose for which it is used in the paper, that is, for storage calculation, but will enable a comparison to be made of the variation of streams, not only in one section or territory, but all over the world.

As the variation of a stream, however, is influenced, among other things, by the area of its catchment, inasmuch as the greater this area, the less variation the stream is likely to have, perhaps the author at some future time could expand this idea to cover the catchment area. This might not be included in terms of the coefficient, but possibly in some other way so that when comparing two or any number of streams, relative to variation, their catchment areas would be brought into the comparison.

In regard to the normal storage diagram developed, this, if the writer understands it correctly, is not based on the actual records of the flow of any one particular stream, but on an artificial record formed from the records of the flow of all the streams used in the analysis. Such an artificial record makes a series covering a period of 300 years, or thereabouts, and may be open to the objection suggested by the author, that it does not reflect any climatic changes which might occur in the actual series, were it not for the fact that such changes or movements are on such a large scale of time that any objection of this kind may be disregarded.

The utility of the normal storage diagram is shown by the fact that by this method the storage required on any stream, to maintain a certain draft, can be determined more accurately than from the actual run-off records of the stream itself, unless perhaps these records covered an exceedingly long term of years, many times longer than the longest now in existence. This diagram method is a great advancement in storage calculation, and very likely will be used as the standard method in such calculations in the future. Even were it the case that actual run-off records could be depended on for storage calculation, this normal storage diagram would be valuable as a check.

A very important point stated by the author is that the storage on a tributary, within certain limits is as useful as that on the main stream. This no doubt is true within the limits stated in respect to reasonably large storage, but is it true in respect to small storage

Mr. Tighe. where the capacity of the reservoir and the draft thereon would be such that the reservoir would empty and fill several times a year or several times in the summer season? For instance, if the capacity of the Borden Brook Reservoir, of the City of Springfield, referred to by Mr. Hazen, was only 70 000 000 gal., would this reservoir, with its 8 sq. miles of catchment, be as efficient as the intake reservoir of the same capacity with its 48 sq. miles of catchment, assuming that the pipe line had a discharging capacity of 20 000 000 gal. per day?

The writer has made no test figures on this, yet it seems to him that the intake reservoir would be the more efficient, as it would be possible for it to fill more often in the summer season, because of its larger catchment, than the Borden Brook Reservoir.

Another point that should not be overlooked, relative to storage on tributaries, is the water that is lost by percolation and evaporation in its course from the storage to the main stream and intake, especially if conveyed intermittently in the old stream bed.

Although considerable attention has been given to the loss of water by evaporation from the surface of reservoirs, it seems that much less has been given to the loss of water by percolation from reservoirs.

When a reservoir is built and filled, the level of the ground-water above the dam is raised and, consequently, some extra storage capacity is provided beyond the apparent capacity of the reservoir, especially if the ground surrounding the reservoir is porous.

On the other hand, because of the ground-water being raised above the dam, if there is any connection between this and the ground-water below the dam, the flow of the latter, owing to the higher head, must be increased, and this increase will come or be fed from the reservoir. It can be argued, of course, that, with the foundations of a dam resting in impermeable material, percolation from the reservoir, under the dam at least, might be considered negligible. The writer would like to get the views of the author and of others on this matter, as he thinks that the loss of water from the average reservoir through percolation is more of a factor than is supposed. If this is the case, it seems that, in storage calculation, especially as other phases of the question are being so closely analyzed, percolation should also be considered.

In conclusion, the writer, under whose supervision the daily flow of the Manhan River, furnished the author, was measured, and who naturally has been interested in and has given some attention to the matter of stream flow and storage, considers this paper the most comprehensive analysis that has ever been presented on the subject. The author certainly deserves the thanks of all hydraulic engineers for the exhaustive manner in which he has treated the subject and the new light which he has thrown on it, especially those phases of it which hitherto have been neglected.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

AN INVESTIGATION OF SAND-CLAY MIXTURES FOR ROAD SURFACING.

Discussion.*

By James Owen, M. Am. Soc. C. E.

James Owen, M. Am. Soc. C. E.—This question of sand-clay roads was initiated about 10 years ago at the suggestion of the United States Office of Public Roads, and the writer believes that roads of that type were afterward constructed in South Carolina. The idea was suggested to the writer by his experience in the construction of a speedway, for the surface of which no natural materials were available, so that they had to be manufactured in the best possible manner, and the mixture that had been made in South Carolina was tried. There was available some very good, greasy, gray clay, which was essential for the work, and about 2 miles away there was a very good bed of sand to mix with it. The writer, however, made five or six mixtures of the sand and clay before he obtained what might be considered a perfectly consolidated surface. These mixtures were put down in places where there was considerable travel, and thus the best method of surfacing was ascertained. Mr. Owen.

The speedway was built with the mixture giving the best results; the clay was spread over it in lumps; the sand was then put on top of that, and the two were spaded together and rolled.

One interesting thing happened in these experiments. The mixture was made according to a preconceived idea of the standard surface, and when it was completed it was impossible to get a smooth trotting way for horses. The surface was rolled, but it would not come down, under the conditions prevailing at that time. By accident, a bed of what is known as New Jersey loam was found, and a coating of this, from $\frac{1}{4}$ to $\frac{1}{2}$ in. thick, was put on, rolled down, and,

* Continued from April, 1914, *Proceedings*.

Mr. Owen. with the consolidation of the gravel and the clay on top, and the addition of the loam, a perfectly smooth and constant roadway surface was obtained.

It is well to note here that, in the construction of sand-clay roads, strange results may be obtained. In New Jersey there are two classifications of roads; in the north they are of stone, in the south, gravel. In both sections there is an enormous amount of travel, automobile and horse, and the experience of to-day tends to confirm the belief that the gravel roads are much more constant in uniform maintenance, much cheaper to repair, and much more satisfactory to travel on than the stone roads, and here comes in the whole question, as enunciated in this paper, that the original idea of macadam roads as the best means of travel under the old regime, of horse travel and hard steel tire travel, has passed away; that in all localities of the United States there is material at hand which should be used for the purpose for which it is required, and manipulated so that the results are good.

Roadmakers in the future must not rely on stone roads to maintain the desired conditions for the wear and tear of travel.

In the case of the Georgia roads, the climate must be considered, and this may be more serious than is thought at first. There the frost does not penetrate to any great depth; there is rarely any frost, except in the high lands; but that does not affect the maintenance of roads, and, as far as that is concerned, the practice that is applicable to Georgia may not be applicable in the Northern States, where frost often penetrates the ground from 2 to 3 ft.

Experience to-day in New Jersey, however, shows that there is no doubt that it is much more economical to use the natural material, properly manipulated, than to import ideal material to make the ideal road, which in the end does not give ideal results.

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PAPERS AND DISCUSSIONS

CINDER CONCRETE FLOORS.

Discussion.*

By Messrs. Arthur H. Diamant and J. R. Worcester.

Arthur H. Diamant, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest, and, having had experience during the past 5 years in the use of reinforced cinder concrete, agrees with the author that the present method of tests of floor slabs does not conform to conditions on construction work. Mr. Diamant.

Aside from the design, however, the great fault in the construction of such floors is the lax supervision in placing the material. The majority of apartment and loft buildings in New York City are erected by speculators. The men in charge of the erection of these structures for the builders know very little about reinforced concrete work, and are rarely on the floors where the arches are being placed. The inspector of the Building Department has so much work to supervise, that he can devote very little time to this part of his duties, and the result is that there is too much dependence on the contractor to do good work. His men, usually unskilled laborers, are required to fill a certain number of centers in a specified time, and, in order to do this, sufficient care is not taken to place the reinforcing metal properly. With the exception of wire-cloth, the spacing of the metal is usually guessed at, and thus its weight per square foot of arch is often insufficient.

Another cause of trouble is in the mixture of the cinder concrete. When hand mixing is done, there is very little attempt to measure the ingredients, and, in machine mixing, the cement, sand, and cinder hoppers are not kept filled. The writer was on the tenth floor of a hotel, being erected under his supervision, when he noticed that the cinder

* This discussion (of the paper by Guy B. Waite, M. Am. Soc. C. E., published in April, 1914, *Proceedings*, and presented at the meeting of May 6th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Diamant. concrete which was being hoisted did not seem to have the proper proportion of cement. The contractor had an efficient mixer plant in the cellar, and when the cause of the poor mix was investigated, it was found that the laborers were so busy keeping the sand and cinder bins filled that they neglected the cement bin.

One of the most pernicious faults is the cutting of holes in concrete arches shortly after they are laid. In an inspection of a large loft building, the writer condemned a panel of reinforced cinder concrete floor which adjoined a stair-well. A large hole had been cut through this panel for a 5-in. soil pipe, and smaller holes for the passage of water, steam, and electric pipes. The reinforcing rods had been cut and bent so that they would not interfere with the pipes. Before the arch could be taken down, it failed because a barrel of lime was rolled over it. There is no excuse for work of that kind, for, in a properly designed structure, the location of all pipe lines is known, and openings can be left in the arches when they are poured.

In conclusion, the writer wishes to state once more that, even though the floor slab is well designed, the careful work of the engineer is not properly carried out because of inefficient supervision in the field.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E. (by letter).—The argument of the author in favor of establishing allowable unit stresses and physical properties of cinder concrete as a means of calculating safe loads for certain designs is sound and convincing. Though there are some difficulties in the way of reaching satisfactory units, they should be overcome if we consider that the margin of inaccuracy will be much less than in any attempt to determine strength by crude, isolated, field tests of actual construction.

The chief uncertainty with regard to the material is on account of the wide range in quality of cinders, running, as they do, all the way from a soft, impalpable ash to a vitrified clinker. The writer has not seen a satisfactory specification for cinders, by which good material for concrete can be distinguished from bad. It is easy enough to write one which will exclude bad material, but the trouble with it would be that it would be so stringent that no material could be obtained which would comply with it. It is to be hoped that further light will be shed on this subject. Another uncertainty in the value of the best cinders lies in the great irregularity in the proportion of fine particles, corresponding to sand, which is contained in every load. This variation is so great in practice that frequently a batch of concrete may contain scarcely any material which could be classed as coarse aggregate. Under such circumstances we would obtain for a product, using 2 parts of sand and 5 of cinders, a 1 : 7 mortar, of which part of the aggregate is less hard than sand. It is chiefly to these peculiarities that we may attribute the wide discrepancies in the results of experi-

ments with cinder concrete, as shown in Table 3, and the general disrepute which the material has among structural engineers. Mr. Worcester.

It is true that cinder concrete slabs, in spite of their disadvantages, have been used to a phenomenal extent, and it is also a fact that failures which can be shown to be due to the use of this material are remarkably few in number. This being the case, it certainly behooves the Profession to deal with the subject in a tolerant spirit, and to do what it can to lay down rules for its safe use.

This Society's Special Committee on Concrete and Reinforced Concrete, in its report presented in 1913 prescribed units, which, though tentative, seem to correspond fairly well with present practice. The subject of the modulus of elasticity of cinder concrete is not referred to in that report, presumably being classed with concretes having an ultimate strength of less than 2 200 lb. per sq. in., for which the ratio of moduli is to be taken as 15. This low value of the ratio does not correspond with general practice. With this exception, the rules laid down in the report cover the method of computation and manipulation quite comprehensively.

Although cinder concrete may prove entirely satisfactory in floor and roof slabs, it is altogether too uncertain a material to recommend for beams, girders, and other structural members, and it is objectionable even for slabs when the span is too great. The reason for this seems to be that the softness of the aggregate allows an inelastic set, which becomes apparent in the deflection when the span is sufficient. It may not be possible to make a hard and fast rule limiting the length of span, for in some places the deflection may not be of importance, or, it might be compensated by cambering the slabs, but the peculiarity should not be lost sight of.

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PAPERS AND DISCUSSIONS

REINFORCED CONCRETE RESERVOIR AND COAGULATION PLANT AT ST. LOUIS, MO.

Discussion.*

By Edward Flad, M. Am. Soc. C. E.†

Edward Flad, M. Am. Soc. C. E. (by letter).—Mr. Finch suggests that the working tensile strength allowed for the concrete, 290 lb. per sq. in., is excessive. He is correct in his statement that Mr. Andrews advises the use of this stress for 1:1:2 concrete. The writer is unable to state definitely at this date the line of reasoning that led to the selection of 290 lb. as a safe stress. Probably the original intention was to use 1:1:2 concrete, allowing the stress recommended by Mr. Andrews, and, later, when it was decided to use a 1:1½:3 mixture, it was concluded that the stress of 290 lb. would be well within the probable ultimate strength of that concrete; moreover an additional factor of safety was provided in assuming a high value for the modulus of elasticity of the concrete. In the structure as built no vertical cracks have been observed, but there is still evidence of moisture at some of the horizontal joints, due probably to a failure to clean the old surface properly before depositing the new concrete. Mr. Flad.

The construction suggested by Mr. Finch and Mr. Potter for the joint between the side and bottom of the tank appeals to the writer as an improvement, except for the difficulty of making this joint water-tight. In the design of the St. Louis Reservoir, the stresses at this point could only be determined approximately, there being a combination of cantilever action and hoop tension, and, in this respect also, because of the extra expense involved, the design is inferior to one containing an expansion joint at the junction of the side with the bottom.

* Continued from March, 1914, *Proceedings*.

† Author's closure.

Mr. Flad. The arrangement for the removal of sludge while in operation, suggested by Mr. Potter, would have been of doubtful benefit for this particular reservoir, as there are two preliminary steel settling tanks, each 75 ft. in diameter and 25 ft. high, through which the water passes and from which the greater quantity of the sludge is removed before the water enters the concrete reservoir; there is no difficulty about arranging for cleaning the reservoir at times when it will not interfere with the operation of the plant, and the proportion of water to sludge required in cleaning is probably less than would be required with the plan of removal during continuous operation.

It is gratifying to note that Mr. Buerger's interesting comparison of designs containing cantilever walls with those providing for tension rings leads to the conclusion that the design selected, to wit, the tension-ring type, was practically justified, his theoretical determination of economical diameter for the reservoir in question being 134 ft., whereas the actual diameter is 150 ft. As Mr. Buerger surmises, the nature of the foundation available was really the controlling feature in deciding between the two types of reservoirs. A uniform distribution of the load was considered to be of paramount importance.

Mr. Buel seems to be in error in stating that, according to the formula given for the thickness of the concrete wall, the ratio of stress in the steel to stress in the concrete is 9 to 1. The formula is supposed to provide a ratio of 10 to 1, and is derived as follows:

Designating quantities and dimensions by the letters used in the paper:

Area of concrete per vertical foot of wall $+ 12\,T - A$,

Hoop tension $= pr = A\,s + (12\,T - A)c$,

$$pr = A\,s + 12\,Tc - A\,c,$$

$$T = \frac{pr - A\,(s - c)}{12\,c}.$$

Assuming that $s = 10c$,

$$T = \frac{pr - 9\,Ac}{12\,c},$$

which is the formula given in the paper.

The description by Mr. Wegmann of the construction and failure of the Stony River Dam emphasizes again the desirability of carrying cut-off walls down to rock or impervious strata. Happily, the failure was not as serious as at first reported, but it furnished a tender morsel for those who, through personal interest, were unfriendly to the Ambursen Company.

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PAPERS AND DISCUSSIONS

THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

Discussion.*

By Messrs. M. J. Butler, Albert Lucius, Henry W. Hodge, Charles Evan Fowler, L. J. Le Conte, N. Petinot, Leon S. Moisseiff, and F. W. Skinner.

M. J. Butler, M. Am. Soc. C. E. (by letter).—The author has once more placed the members of the Society under obligations. He has dealt most ably with a matter of great interest and importance. Mr. Butler.

It is an error to assume that the limit has been reached with ordinary carbon steel—undoubtedly, if the customer is willing to pay the price, the steel maker will supply a much better quality of steel:

(*a*) By fluid compression of the ingots;

(*b*) By careful heat treatment and lighter and slower blooming, putting more work on the bloom;

(*c*) Cropping away all segregated, piped, and inferior metal;

(*d*) Careful re-heating of billets, slower rolling, and better work generally on the rolls and sections;

(*e*) Such a quality of acid steels—possibly with a cheaper alloy than nickel—would allow of higher unit stress in large members, where shock and impact would be amply cared for.

Alloy steel of nickel and other materials may be secured to meet any reasonable requirements. Steel makers will rise to the occasion, when the demand justifies the outlay in getting ready for steel of such a quality.

In very great bridges, such as are under consideration in this paper, the increased price of the metal would not be so serious; there are too

*This discussion (of the paper by J. A. L. Waddell, M. Am. Soc. C. E., published in March, 1914, *Proceedings* and presented at the meeting of April 15th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Butler. many other factors that go to make the cost, such as the substructure, the erection of great cantilevers, and the special tools, plant, and shop, to fabricate the member. The engineering and the financial costs are to a considerable extent independent.

Mr. Lucius. ALBERT LUCIUS, M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest. Its objects are stated to be: First, to give the weights of different span lengths for simple spans and cantilever bridges, using metals of various elastic limits; secondly, to indicate the extreme practical limits of span lengths of cantilever bridges constructed of such metals; and, thirdly, to demonstrate the comparative economics of finished bridges involved in using metals of the various assumed elastic limits.

Concerning the weights of long-span bridges built of assumed metals of higher elastic limits, whatever their actual amount may be, there is little doubt that weights would reduce about as formulated in the paper if the unit stresses follow the elastic limits, as they should, provided the metals have the same superior properties as to ductility which they have as to strength, for then only would confidence in the material be justified to the extent of developing all sections and all details carrying calculated stress by the proportional unit strains. Members and their connections and all details, the function of which is to distribute local strains over sectional areas, could then be made of the same material working under the same stress and elastic condition, but, if the high alloys are of inferior ductility and connections and distributing details must be made of softer and weaker metals, and the unit amounts of extension and compression vary in the different parts forming the ends of a member, the difficulties of making proper connections would be increased and the advantages of reduced main sections would be partly neutralized.

Concerning the extreme length of cantilever spans, it is the writer's opinion that it has been reached, as far as practicable; that it will not be carried very much further, even if higher grade material is available; and that every consideration of reliability to carry load, simplicity of the carrying member, availability of the highest grade of material, and ease of construction and erection, points to the suspension bridge as the type to take care of extreme span lengths where bridges are preferred to subway constructions.

Concerning the economics involved in the use of high elastic limit materials, these are expressed most readily in a sufficiently satisfactory way by the ratio of elastic limit stress to price per pound of the metal, and whichever metal sells at the least price for the highest elastic limit, all other properties being in proper relation to it, is the cheaper material, and this would apply to the various types of bridges alike as far as their fabricated cost is concerned. Generally,

the idea of using simple spans up to their extreme limit of possible application cannot be attractive to engineers, because there is hardly any other reason for long spans than the requirement to maintain an unobstructed right of way under these proposed spans. The feasibility and cost of erection will determine the type, and a cantilever or continuous span over several openings, or an arch bridge, all of which can be erected cantilever fashion, would probably be selected before the economic limit of the fabricated simple-span bridge is reached. Besides, in long-span bridges, especially where prominently placed, esthetic considerations do and should enter more emphatically and obscure the importance of relative economics still further. Mr. Lucius.

Concerning the desirability of improving on the present available bridge material, both as to uniformity of present properties and in regard to increase of all its elastic properties and its ductility, there is probably no difference of opinion among engineers, and each, no doubt, does all he can to encourage and help push manufacturers to favorable solutions; but this is another subject, and in the care of the metallurgist. It is hardly likely that it is inertia alone which makes progress slow and apparently does no more than maintain a stationary level. It is more likely that there is a greater possibility of making economic improvements on the present grade of steel by cheapening it and making it more uniform in quality, than to make a higher grade of steel at a price which will give it an advantage commercially, except for special purposes, in which case the price is not so important relatively to the whole special purpose to be accomplished. The vast majority of railroad bridges can be well taken care of with the present grade of steel, and whether lighter sections of higher grade steel worked to higher strains would be an all-round improvement for the bulk of bridgework, might be open to debate. It certainly would not be the case, in the writer's judgment, if higher strength did not also carry with it higher ductility.

The engineer must get next to the metallurgist. It is highly probable that any metallurgist who knows how to produce a better steel alloy at commercial value will succeed very promptly in getting it on the market, and engineers would accept it as promptly and adjust their constructions to it.

Henry W. Hodge, M. Am. Soc. C. E.—The length of bridge spans in general use has been increasing steadily, and we have reached limits where the dead weight of the structure has become the largest portion of its carrying capacity, so that some method of keeping the weight down is a necessity for the construction of the great spans now contemplated. The only way to reduce the dead load materially is by the use of metal of higher carrying capacity than our present materials, and a long step in this direction has been made by the use of nickel Mr. Hodge.

Mr. Hodge. steel, which has 50% greater carrying capacity than the carbon steel in general use.

The trusses of the three 668-ft. spans of the St. Louis Bridge were designed for nickel steel throughout, except certain minor sub-members. Nickel-steel eye-bars and carbon-steel compression members were also used, the floor system and bracing being of carbon steel in each case.

The weights of each span were:

With complete nickel-steel trusses......... 9 200 000 lb.
With nickel-steel bars, and the rest of carbon steel ....10 900 000 lb.

The dead load of railways, tracks, etc., was 5 500 lb. per lin. ft., so that the total average dead load was:

With nickel-steel trusses............19 300 lb. per lin. ft.
With nickel-steel bars, and the rest of carbon steel ....21 800 lb. per lin. ft.

Thus, the nickel-steel compression members in the trusses made a difference in weight of 2 500 lb. per lin. ft., or 13 per cent.

The average live load on the two decks was equal to 16 600 lb. per lin. ft., thus the use of nickel-steel compression members made a saving of 7% in the total load on the structure.

The average unit prices for the two classes of material in this structure, erected in place, were:

Nickel steel, 5.6 cents per lb.
Carbon steel, 3.95 " " "

making a difference of 1.65 cents, or 42% of the price of the carbon steel; but the elastic limit required for the nickel steel was 50% higher than for the carbon steel, so that the nickel steel was the cheaper, strength for strength.

This steel had $3\frac{1}{4}$% of nickel, but manufacturers are now commercially producing an alloy steel, with not more than $1\frac{1}{2}$% of nickel, together with small percentages of chromium and vanadium, which has all the properties of this steel, at a very much reduced price, so that there is at present a readily obtainable material, which is 50% stronger than the carbon steel in general use, at a comparatively small increase in cost.

This increase of elastic limit to 50 000 or 60 000 lb. per sq. in. will help greatly in the construction of spans of considerable length; but, for the very long spans now being planned, a still stronger material is needed, and can economically be used at a very considerable increase in price. In the design for the 2 880-ft. suspension span for the North River Bridge at New York, there is great economy in placing the stiffening trusses along the cables, so that articulated

joints are required, thus necessitating eye-bars in place of wire, which has heretofore been used in long suspension bridges. Mr. Hodge.

The total live load on the eight tracks, two roadways, and two sidewalks, is 20 000 lb. per. lin. ft., when all are completely loaded; and, if the structure were made of carbon steel, the dead load would be about 110 000 lb. per lin. ft., which would make the sections almost prohibitive. The designs, therefore, have been based on the use of eye-bar cables of alloy, heat-treated steel, having an elastic limit of not less than 120 000 lb. Such steel has already been manufactured for limited sizes, and as it is only here required for eye-bars, from which any number of full-sized specimens can be tested to destruction, there will be no doubt as to whether the strength and other qualifications are fulfilled. With such material, the total average dead load of the structure is 64 000 lb. per. lin. ft., so that this material makes a saving of 46 000 lb. per lin. ft., or practically 50% of the total load on the structure.

Such a material will naturally cost a considerable price, but it will be about four times as strong as the usual carbon steel; and even at much more than four times the cost, it would make a saving, on account of the large decrease in dead load.

About 40 000 tons of such bars will be required, practically all duplicates, so that there is little doubt that they will be furnished at a price which will make a great economy in the structure.

The speaker, therefore, fully agrees with Mr. Waddell that such high-value alloy steels are a necessity for coming bridge structures, and has the fullest confidence that our metallurgists will meet the demands as they arise.

Charles Evan Fowler, M. Am. Soc. C. E. (by letter).—For many years the writer has known of the higher grades of steel produced by European manufacturers, but, owing to the great demand on the mills of the United States, it has been a case of taking what is offered by the manufacturers or paying the extras asked for nickel steel or anything except the ordinary grades. The tariff conditions will now allow the importation of European high-grade steels for Pacific Coast fabrication, but, until shops of sufficient size are established there, little advantage can be taken of such foreign products. This possible source of supply may eventually result in American mills being willing to meet the demand for nickel or other high steel alloys at a reasonable price. Mr. Fowler.

In the writer's discussion* of Mr. Waddell's former paper, "Nickel Steel for Bridges", he called attention to the possible use of ferruginous nickel as a cheap means of obtaining the nickel ingredient for nickel steel, and it is gratifying to know that this has been declared possible

* *Transactions*, Am. Soc. C. E., Vol. LXIII, p. 300.

Mr. Fowler. by competent metallurgists and at a reasonable cost; so that now it seems to be only necessary to find philanthropic steel manufacturers who will make only a reasonable charge for manufacturing plates and shapes of the desired composition.

To conduct the extensive experiments which are desirable on nickel steel and other high steel alloys, it will certainly be necessary to enlist a really enthusiastic support from the mills before anything satisfactory can be accomplished. When it is possible to do this, and to raise the necessary funds, the work should be carried out under the direction of this Society, acting in conjunction with the American Society for Testing Materials.

The great amount of work done by Mr. Waddell, as shown by this and his former paper, will entitle him to a large share of the credit for the great spans which may be built in the future if, as a consequence, higher steels are made possible.

The diagrams of weights of metal in bridges, for both the usual and the higher grades of metal, show conclusively what one would naturally infer as to the great saving that may be made by the use of a steel of high elastic limit which can be obtained at a reasonable cost. It is to be regretted that all such investigations are not made on a common basis, both as to loadings and specifications, so that the work of various engineers along similar lines may be more readily compared, and the desired end be more quickly reached. To this end, the proper committee of the Society should formulate rules for all such investigations and calculations.

The engineer who has been in direct touch, not only with structures manufactured from his own designs, but also with those of hundreds of other engineers, as has been true in the writer's case at the shops of the Youngstown Bridge Company and at other large shops, and throughout a wide experience of more than a quarter of a century, will realize that the personal equation of the designer does result in such a wide variation in weights that, unless each one is tied down to the hard and fast rules of the same specifications, the resulting data cannot readily be compared, especially for long spans.

The real comparison can only come from the various designs that would naturally be made for any specific location and the investigation of the composition of the necessary high-grade metal required for that particular structure, as was the case for the St. Louis Eads Bridge. A structure costing from \$30 000 000 to \$60 000 000 would easily stand the charge of \$100 000 or more for such experiments.

Long spans are seldom contemplated or built from motives of economy, but are the result of the necessities of commerce, of finding good foundations, or as necessary connecting links in lines of traffic and communication, regardless of whether the structure in itself will

be a paying investment. When, from some cause or other, long spans are found to be necessary, the engineer must determine the class of structure that will be possible: Mr. Fowler.

First.—To carry the class and quantity of traffic to be imposed or cared for;

Second.—That will be possible, due to the foundations that can be obtained;

Third.—That will be possible, due to the material that can be obtained from which to fabricate it;

Fourth.—That will be possible, owing to the limitations that may be imposed by the methods and means of construction;

Fifth.—That will be a paying investment from a dividend-paying standpoint, if that be necessary;

Sixth.—That will at least be possible from related financial conditions.

These factors are correlated to such an extent that a very wide investigation would be necessary, in order to set the limits of span and expenditure.

Recent researches seem to indicate:

That simple spans can be constructed by using nickel steel to a greater or less degree, up to lengths of from two to three times those which have been built;

That suspension spans and cantilevers will reach about the same cost at some span length between 1 600 and 1 800 ft.;

That, by the use of nickel steel to a large extent, suspension spans can be built economically, where the traffic is sufficient, up to about 3 000 ft. span;

That, by the use of nickel steel to a large extent, cantilevers can be built economically, where the traffic is sufficient, up to about 2 500 ft. span.

However, inasmuch as the dead weight of such structures is very great in proportion to the live load, we can largely disregard impact, and use correspondingly high unit stresses, thus allowing at present cost the use of nickel steel for all the suspended structural parts of a suspension span, thereby making such a structure, with its high steel cables and comparatively low erection cost, the best to adopt for long spans, from every point of view, beyond a span length of about 1 700 ft. Such a structure will carry satisfactorily all classes of traffic, and satisfy most fully all the six requirements previously given.

The question of right of way and terminals may add so greatly to the cost of the project as to make its realization impossible.

In his discussion of Mr. Waddell's former paper, the writer called attention to the various cost factors that would come in to determine

Mr. Fowler. the possible span length of any type, or indeed the type to be used in a particular location. The sub-factors of erection cost will also serve to decide these things. The empirical erection cost formula, devised by the writer some years ago, will serve to illustrate this point, there being six different factors which enter into the cost of only this one portion of the amount necessary to expend for the construction of a bridge superstructure:

$$C = a + \sqrt{l} + \frac{3}{4} h + \frac{200}{d} + 5p - \frac{1}{5} \sqrt{w - 500}$$

C = cost of erection, in cents per 100 lb.;
a = a constant for each type of structure;
= 15 cents for railway pin trusses;
= 25 cents for railway riveted trusses;
= 20 cents for railway girders;
= 20 cents for highway pin trusses;
= 30 cents for highway riveted trusses;
= 15 cents for highway girders;
l = span length, in feet;
h = height of falsework, in feet;
d = daytime temperature, average, in degrees Fahrenheit.

Taking the case of a riveted railway span 225 ft. long, 48 ft. height of falsework, average temperature, 40°, 2 coats of paint, and weighing 2 100 lb. per lin. ft., we find the probable erection cost to be 83 cents per 100 lb., or $16.60 per ton.

A railway pin span of 144 ft., 36 ft. height of falsework, average temperature, 50°, 2 coats of paint, and weighing 1 400 lb. per lin. ft., erection cost equals 62 cents per 100 lb., or $12.40 per ton.

These two examples show what an influence a change in any one of the factors will have on the unit erection cost. For cantilevers of the type of the Poughkeepsie Bridge, or the Knoxville Bridge, designed and built by the writer (Fig. 24), every other span being a fixed or anchor span, the bridge must have falsework, so that, for a bridge of this type one can take the coefficient of h at only $\frac{3}{8}$ instead of $\frac{3}{4}$. The formula was deduced to fit certain conditions, which to a large extent were due to the personal equations of the designer and the erector; and, with plans prepared by some designers, the cost of erection would exceed very greatly the values found from the formula, it being only too common on the part of many designers to forget that structures must be erected at a reasonable cost, and still others seem to forget the process of erection entirely.

Many erection costs, of course, will exceed greatly what they should, due to unforeseen causes. The White Pass, Alaska, Arch constructed

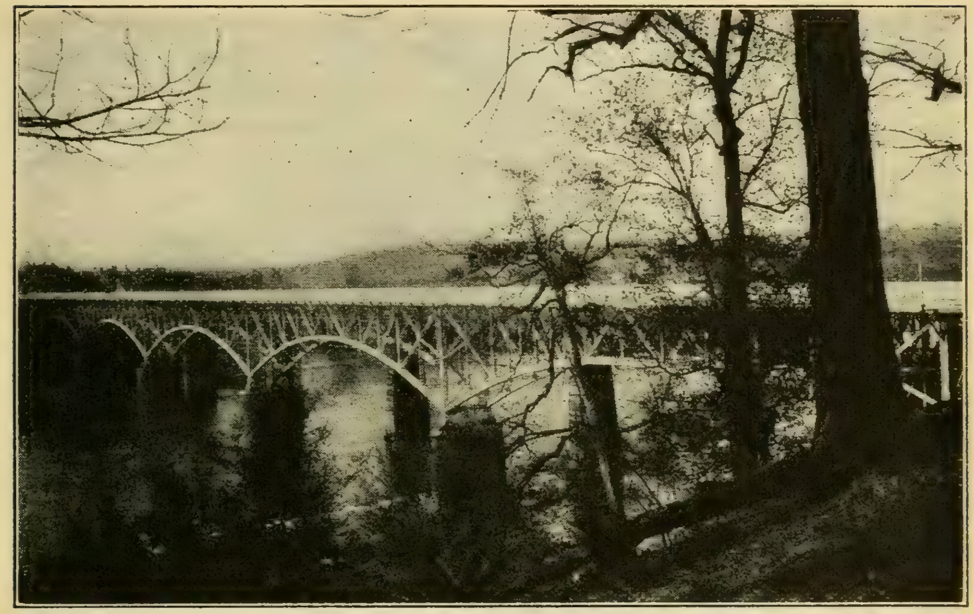

FIG. 24.—KNOXVILLE STEEL ARCHED CANTILEVER BRIDGE.

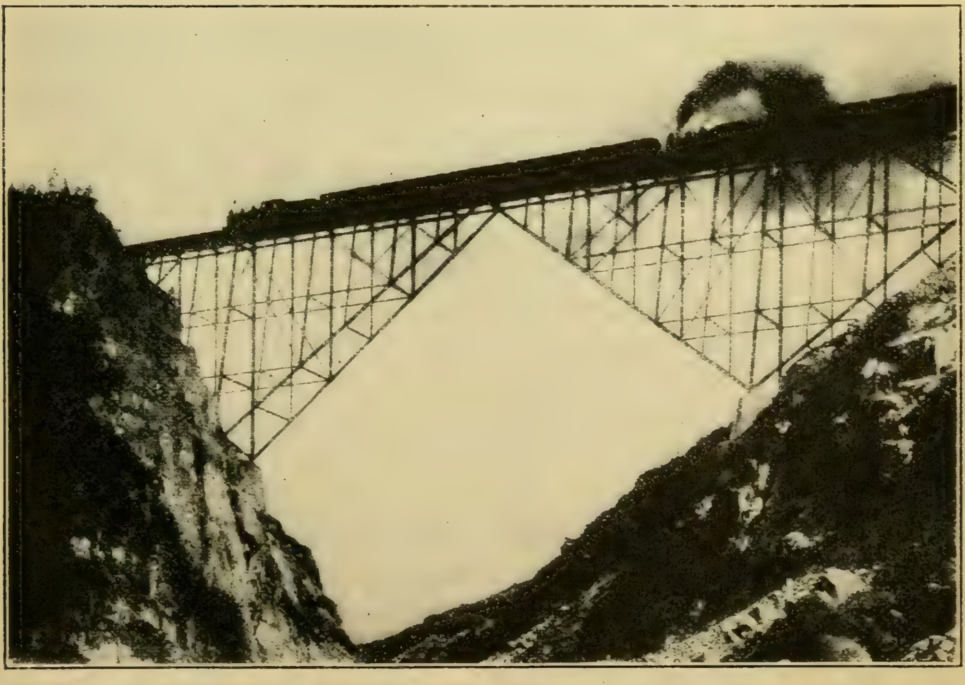

FIG. 25.—SWITCHBACK ARCH. WHITE PASS AND YUKON RAILWAY, ALASKA.

Mr. Fowler.

by the writer, was to have been completed before the winter season, but, owing to a strike of the workmen in September, and no telegraph line available to obtain a new crew from Seattle, 1 000 miles distant, some 6 weeks were lost, and the structure was completed by the men working in two shifts of 2 hours each, one crew warming up in camp while the other worked. The arch was completed in December, and is shown in Fig. 25. Had it not been for the delay and consequent extra cost, the erection could have been carried out on a very economical basis, as the falsework necessary for the anchor arms was very slight, and one top traveler carried out the entire work of erection.

The extra cost due to delay in this or other cases, cannot, of course, be considered in making a design, but there are many features of erection methods which should be studied out during the original investigations. So far as the writer is aware, no scheme of erection was ever formulated for the construction of the Williamsburgh Bridge over the East River, on which he was Consulting Engineer for the erection of Manhattan tower and approaches. The Brooklyn tower erection had been planned and partly carried out by constructing trusses across from masonry pier to masonry pier, and these supported a temporary combination tower about 300 ft. high from which to erect the steel tower.

For the Manhattan end the tower was erected up to the floor level by stiff-leg derricks on a heavy bent at the end of the approach-span falsework. The latticed strut and floor-beam girder in the tower was also placed, and on this a timber tower about 150 ft. high was constructed to use in erecting the remainder of the tower steel, thus saving some thousands of dollars in erection cost.

The foregoing will serve to make perfectly plain the writer's reasons, in some few respects, for regarding diagrammed weights and costs as only very general proof of the necessity, in any particular case, for the use of the highest obtainable grade of steel. There are items in every part of the cost, of both substructure and superstructure of any great bridge, which will vary widely from similar items of cost in another large structure, and it is only by extensive investigations at each particular location that we can arrive at a close semblance of the truth and avoid the errors which often wreck meritorious projects.

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is highly pleased with this valuable paper. The practical results given are exceedingly encouraging for future developments. The writer would respectfully suggest to the experimenters that they also try a small batch of aluminum steel. The main objection to its use, heretofore, has been the high market price. Lately, however, the price has come down to 20 cents per lb. in large lots. It is his firm belief that if the ordinary steel is first "thoroughly purified", and its elastic limit,

Mr. Le Conte. thereby, raised to 60 000 lb. per sq. in., and then a cake of aluminum, say 3%, be added to the molten metal, the result will be a highly satisfactory alloy, and at no great increase in cost per pound. The aluminum not only strengthens the steel enormously, but also toughens it and wipes out all brittleness, and thus renders all general shop work much easier and more satisfactory and reliable in every way.

In case some of the material should be condemned as not coming up to the requirements of the specifications, the condemned material could easily be turned over to the wire works which would be only too glad to get it, as this aluminum steel makes the finest kind of high-grade wire, suitable in every respect for suspension bridges, as it is exceedingly strong, tough, free from brittleness, and non-corrosive to a high degree. Ordinary sections of this alloy will have an elastic limit of 100 000 lb. per sq. in., and the wire possibly 200 000 lb. per sq. in.; and at a cost probably not exceeding that of ordinary carbon steel by more than 2.5 cents per lb.

Mr. Petinot. N. Petinot,* Esq.—The speaker agrees with Mr. Waddell that the first step is to experiment on the purification of steel so that the metal may be brought to its maximum efficiency. He cannot see why it should not be possible to secure regularly physical properties within a limit of 5% in a steel carrying predetermined quantities of carbon and nickel, for instance.

The speaker has been experimenting extensively, both in France and in the United States, with the manufacture of alloy steels, and particularly with nickel steels of the same supposed analysis, and has found that very often two melts of open-hearth nickel steel of the same analysis, when physically tested, have shown a variation of 15% or more. Carrying the investigation further he found, by microscopic examination, a large difference between the two steels. The poorer very often showed very large colonies of manganese sulphides, iron silicates, and slag. These are weakening elements in steel of any grade, even if it is an alloy of plain carbon metal.

When the manufacturers of steel find the way to remove such elements, it will be possible to secure the maximum physical qualities from each component of every grade of steel, and there should be no great difference between any two steels of the same chemical composition.

Increasing the carbon content in steel of any grade is a cheap way to increase the alternate strength, and, according to the speaker's experience, a satisfactory way, provided the steel has been thoroughly cleaned and the segregation reduced to the minimum.

In reference to the possibility of producing a nickel steel by the use of a ferro-nickel containing 10% of nickel, this problem presents

* Metallurgist, The Titanium Alloy Manufacturing Company, Niagara Falls, N. Y.

Mr. Petinot.

no difficulties at all, as such steels have already been produced in 5-ton melts. A few years ago a pig iron was produced by smelting pyrrhotite—containing nickel—in an electric furnace. This pig iron contained from 4 to 5% of nickel and was low in sulphur and phosphorus. By varying the proportions of such pig iron in the mixture used in making nickel steel by the open-hearth process, it will be possible to obtain any nickel content actually desired.

It will be noticed that, in the making of nickel steel, the nickel is usually added with the cold charge, and does not, therefore, oxidize during the periods of melting and finishing the steel, and its use as a component of pig iron would be the same under similar conditions.

The speaker believes that, by experimenting thoroughly with ferro-nickel in the manufacture of nickel steel, it will be possible to get a product which will be identical with that obtained by the use of metallic nickel, and at a much lower cost.

Mr. Waddell has asked what effect the use of titanium might be expected to have on the steel. Titanium has a great affinity for nitrogen and oxygen, so much so that, prior to this time, it has been very difficult to obtain a steel with a content of more than 0.25 of that metal, and such steel showed no particular advantages over one which had been treated with 0.10 of titanium, and after such treatment carried only 0.02 to 0.03 of the latter.

Titanium is on the market commercially in the form of ferro carbon-titanium, containing from 15 to 20% of titanium and from 6 to 8% of carbon. It has been found that a ferro-titanium, with a higher titanium content, has too high a melting point for general use.

Titanium, because of its affinity for nitrogen and oxygen, the latter either in the form of a gas or an oxide, is to be considered as a powerful deoxidizer and scavenger. It is the only deoxidizer known at present which can be used without danger of leaving any of the products of its oxidation in the bath of steel.

In the treatment of steel with titanium, the ferro carbon-titanium is always used as the last addition to the steel after it has been tapped into the ladle, that is, after manganese, silicon, or other alloys, if any, have been added. Titanium will react first on nitrogen (always present in a greater or less quantity), forming titanium nitrides (Ti_2N_2), which rise to the top of the ladle. It will then act on the oxygen in solution in the steel, and on oxides of iron, manganese, etc.

Particular attention is called to the presence of manganese oxide in steel. Ferro-manganese is added to the steel, not only as a deoxidizer, but also to furnish sufficient manganese to combine with the sulphur, giving manganese sulphides, which are less brittle than iron sulphides. It has always been found that the rolling properties of steel are enhanced by a small manganese content, and it is a well-known fact

Mr. Petinot.

that it is impossible to roll a steel ingot which does not contain manganese.

Now consider, for instance, what happens when manganese is added to molten steel. The iron oxides, for instance, are robbed of their oxygen, and manganese oxides are formed, as shown by the following chemical equation:

$$FeO + Mn = Fe + MnO,$$

but, as manganese oxide will partly remain in the steel, if a piece of this steel is analyzed, the chemical laboratory will report so much manganese, but there will be nothing to show in what form this manganese occurs, whether as an oxide, sulphide, or some other compound.

It is difficult to determine whether a remainder of manganese oxide in the steel is more or less harmful than a content of iron oxide, which it may have replaced. In the manufacture of soft steel, with a carbon content of from 0.08 to 0.10 and manganese content of 0.40, it has been proved that, by eliminating the manganese oxide completely, no blisters will be found when such steel is rolled into sheets and galvanized.

When titanium is added to steel it will react as shown by the following equations:

On nitrogen:

$$Ti_2 + N_2 = Ti_2N_2 \ldots\ldots\ldots\ldots (1)$$

On oxygen:

$$Ti + O_2 = TiO_2 \ldots\ldots\ldots\ldots (2)$$

On iron oxide:

$$2FeO + Ti = 2Fe + TiO_2 \ldots\ldots\ldots\ldots (3)$$

On manganese oxide:

$$2MnO + Ti = 2Mn + TiO_2 \ldots\ldots\ldots\ldots (4)$$

The titanic oxide (TiO_2) formed by any of the last three reactions will flux all particles of slag invariably suspended in the steel and carry such slag to the top of the ladle, which will enable steelmakers to teem into the ingots a product, practically free from nitrogen oxides and slag, which are the weakening elements in steel of every grade.

Another function of titanium is to reduce segregation to its minimum. The following explains briefly how segregation is produced: Steel must be considered as a mixture of pure iron, iron carbide, manganese sulphides, iron silicides, iron phosphides, etc., each of the components having different specific gravities, and varying melting points. Now, consider what will happen when an ingot mould is filled with molten steel: The wall of the mould will act as a chill, and the component having the highest melting point will be the first to solidify. If drillings are taken, starting from the outside of the ingot toward its center, it will be found that the carbon and other elements of lower specific gravity and higher melting points will be

higher inside the part cross-hatched in Fig. 26 and also at the top of the ingot. Mr. Petinot.

Now, consider why the various elements composing the steel will segregate, and why the maximum segregation will be found at the top of the ingot. This segregation is produced by the presence of oxides in the steel, no matter whether they are of manganese, of iron, or other elements. Take, for instance, a molecule of oxide, *R* (Fig. 26). The carbon of the steel will react on this molecule, giving carbon monoxide, according to the following equations:

(In the case of iron)

$$FeO + C = Fe + CO \qquad (5)$$

(In the case of manganese)

$$MnO + C = Mn + CO \qquad (6)$$

The tendency of this carbon monoxide gas is to release itself from the steel by rising to the top of the mould. The bulk of this gas has been produced in that part of the ingot which is last to solidify, as previously mentioned. This part of the ingot is composed primarily of iron carbides, iron phosphides, manganese sulphides, etc., so that all these elements will be found in excess near the top.

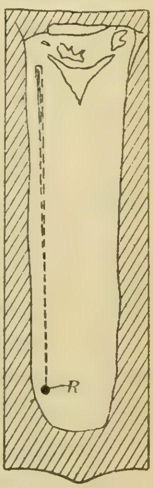

FIG. 26.

It very often happens that a steel which shows in a ladle test 0.40 carbon will show, when rolled into bars, a variation of carbon between 0.35 and 0.45. If titanium is added, it will prevent, to a great extent, the formation of CO in the moulds, because, as already shown by Equations 3 and 4, the titanium will react on these oxides in the ladle, so that, when the steel is poured into the moulds, the only segregation that will occur is that caused by the chilling action of the moulds.

The speaker's experience has been that a clean product, with very little segregation, can be obtained by the use of titanium, at an additional cost of from 25 to 50 cents per ton of steel treated, and, if this is so, the uniform composition of such a steel will easily warrant small additional expenditure.

Leon S. Moisseiff, M. Am. Soc. C. E.—This paper is most interesting and also most timely. Mr. Moisseiff. It attempts to find a way to meet the ever increasing demands made on the ingenuity of the engineer and the resources of the capitalist by heavier loads and greater spans. There is no doubt that the author points the right way into the future: there is a need of a material of a higher strength than that of the steel generally used in bridge building to-day. Whether it be nickel steel, as recently used in some of our long-span bridges, or another kind of an alloy steel, still better meeting the requirements of bridge building, there is a technical need of a higher strength steel. Not only would

Mr. Moisseiff.

heavy railroad bridges with their increasing freight loads be benefited by the higher steel, but also highway bridges of the longer spans.

The advent of heavy automobile trucks has made it imperative to provide heavy floors for bridge roadways. The old rule, of an inch depth of plank for every foot of span for the usual timber floor, died quite some time ago. Nowadays, concentrations of 10 000 to 15 000 lb. have to be taken care of on city highways. Even the old reliable buckle-plate, built in a suspended position, appears to have lost its good behavior. Recently, it has been found on some highway bridges that the buckle-plates are not sufficiently rigid, and that the pavement fails in consequence. This can readily be explained. The suspended buckle-plates are stiffened by the filling of concrete placed on them. The stiffness thus provided was sufficient for the old-time concentrations from wheel loads, and the pavement was supported rigidly enough to give good results. The increased concentrations of modern traffic, however, overcome the stiffening resistance of the concrete and the buckle-plates, and the curve of the plate is distorted under the advancing action of the load. This results in the crushing and crumbling of the concrete in the trough, and the ultimate failure of the pavement.

To insure the good behavior of a pavement, not only a strong but also a stiff support is necessary. Such support will be furnished by a heavy timber floor with floor stringers at close intervals, or by reinforced concrete slabs. In either case a floor system will be required which will have considerable weight. To take care of this additional weight and load a high-strength steel is wanted.

The author has gone to a great deal of trouble to show the economical advantage of higher steel, and there can remain no doubt as to the desirability of its use for bridges.

Granted, then, the desirability of the higher strength steel, questions of design may well be considered. One of these questions may be: what will be the effect of the higher steel on the stiffness and stresses of bridges built of that material?

The higher elastic limit of the new steel will be utilized, of course, for allowing higher unit stresses, which is the purpose of its use. The coefficient of elasticity being practically constant for kindred steel, the higher unit stresses will result in greater elongations of the individual members and greater deflections of the entire structure. The increased deformation of the trusses means a greater deviation from the original truss-form, and will result in much increased secondary stresses.

Assuming, as the author has apparently in the paper, that the allowable unit stresses will retain the same proportion to their corresponding elastic limits as they have in the present practice in the case of carbon steel, the allowable stresses for the hoped for high-alloy steel of 100 000 lb. elastic limit will be nearly three times those of carbon steel with its elastic limit of 35 000 lb. Evidently, the moving load deflections will

Mr. Moisseiff.

become considerable and the secondary stresses may attain the rank of primary stresses. How, then, to take care of the secondary stresses in the erection of the bridge, and provide for the deformation of the trusses under moving load, will raise new and serious problems in bridge design.

Another question is how to connect efficiently the high steel members, apart from using pin connections. With common soft steel rivets the splices would be of excessive length, the efficiency of which is subject to much doubt. Consequently, high-steel rivets must be used. Nickel steel members have been designed by the speaker for corresponding nickel steel rivets, but these rivets have not always proved very satisfactory. They are not easily well driven and are difficult to remove. This refers especially to field rivets which have to be driven with a gun or a machine of limited pressure. Now, if nickel steel rivets having an elastic limit of 50 000 lb. present some difficulties, what will rivets of, say, 80 000 lb. elastic limit offer?

Mr. Skinner.

F. W. Skinner, M. Am. Soc. C. E.—The speaker, having recently had an opportunity of seeing some phenomenally large carbon steel rivets, made for the new Quebec Bridge, would like to confirm to some extent the views of the author and of Mr. Hodge. Some of these rivets—those for the cantilever arms—were 7 in. or more in length, with a diameter of 1¼ in.

Some experimental work was done in which these rivets were specially designed with a slight taper at the points. They were quenched at the points in cold water. When driven, they were held in place by a pneumatic bucker, and the driving was done with a pneumatic hammer. The pneumatic bucker was a special tool, having a hollow cylinder in which a hammer was arranged. The hammer and cylinder were controlled independently, which enabled the bucker to be held in position, and then when the ordinary hammer was operated on the point of the rivet, the secondary hammer in the bucker was operated, and the rivet was thus driven simultaneously from both ends. The result was that they were exceedingly well driven. A row of rivets was driven partly by yoke machines and partly by the hammer and bucker process, as in the field. When they were cut in two, it was almost impossible to distinguish between them, the field-driven rivets being, for every practical purpose, as good as those driven by machine in the shop. Therefore, it seems quite reasonable to expect that field-driven rivets can be made substantially as good as shop rivets, and, if nickel steel rivets can be driven satisfactorily, it may go a long way toward producing equally good results in the field.

Regarding the possibilities of high alloy steel in bridges, there can be no doubt that the need of it is great and the prospect is alluring. The speaker supposes that it is generally agreed that there is difficulty

Mr. Skinner. in securing uniformity in its production. A great many years were required to produce carbon steel which was trustworthy and uniform, and it will no doubt take as long to get alloy steel in uniform condition. The speaker would be very glad to hear from the author about the possibilities of developing a super-strong alloy steel by subjecting finished members to heavy test loads at the shops. It has been pretty well demonstrated that both the elastic limit and ultimate strength of carbon steel, for both tension and compression, can be thus greatly raised. The speaker does not know that this has gone farther than a demonstration, but it was considered more or less seriously for the Quebec Bridge, and perhaps for the St. Louis Municipal Bridge. Such treatment should achieve the double result of increasing the elastic limit and ultimate strength, and therefore the working capacity, and should serve as a valuable test or proof load, disclosing all serious defects.

If an increase of from 20 to 40%—possibly even more—can be practically attained at moderate expense for carbon steel, what would be the result if applied to nickel or other alloy steel? It may be inferred that important results can be obtained. The speaker thinks that it opens up a wide field of conjecture.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE ALFRED NELSON, M. Am. Soc. C. E.*

DIED JUNE 3D, 1913.

George Alfred Nelson, the son of George and Abigail Marion Bigelow Nelson, was born in Lincoln, Mass., on September 20th, 1852. He attended the Lexington District schools and the Lincoln High School and was graduated from the latter in 1872. In 1873, he entered the Massachusetts Institute of Technology from which he was graduated in Civil Engineering in 1877. During a summer vacation, from June to September, 1875, he served as Rodman on the survey and construction of the Boston, Concord and Montreal Railroad, and as an Assistant in the office of the City Engineer, at Concord, N. H.

After his graduation from the Massachusetts Institute of Technology, Mr. Nelson spent two years at his home in Lincoln, and was engaged in surveying and various engineering works in that vicinity. In 1879, he went to Lawrence, Mass., where he was employed as sketch maker in the Designing Department of the Pacific Mills. He remained in this position until August, 1883, when he resigned to become Assistant Engineer in the office of the City Engineer at Lowell, Mass., which position he held until his death.

As Assistant Engineer, Mr. Nelson had charge of the design and construction of several bridges in Lowell, one of which was the Taylor Stone Arch Bridge across the Concord River, the location of which involved difficult foundations, and its successful completion showed the thought and skill devoted to its design. He also had charge of a complete survey of the city for assessors' maps; of all water-works improvements; the extension of the sewerage system into new territory; and the design and cost estimates for the abolition of grade crossings within the city limits.

Mr. Nelson was a man of strong character and his personality and keen mind impressed all with whom he came in contact. He had never been very strong physically, and his constant and close attention to the details of his work affected his health so that he was forced to seek rest at frequent intervals. His death occurred on June 3d, 1913, after an illness of a few months. Mr. Nelson had never married. He is survived by a sister and two brothers.

He had always shown much cleverness with his pencil and crayon, and was an expert photographer. He had shown his photographs at

* Memoir prepared by the Secretary from material on file at the Society House.

various exhibitions in the United States, receiving many medals for his artistic work. With a few others, he was selected to represent the United States at an international exhibition at Berlin, Germany, where his photographs won for him a silver medal.

Mr. Nelson was an active member of the Eliot Congregational Church Society, at Lowell, and for many years was President of the John Eliot Literary Society, contributing greatly to the success of its work by his active energy and personality. He was a member of the Alumni Association of the Massachusetts Institute of Technology, the Technology Club of the Merrimac Valley, of which he was an active president for several years, and the Association of the Class of 1877 of the Massachusetts Institute of Technology. He was also a member of the Boston Society of Civil Engineers, having designed the pin which was adopted by that Society. He was devoted to outdoor sports and took an active part in the snowshoe trips of the Appalachian Mountain Club, of which he was a member. He was also a member of the Vesper Country Club, at Lowell, and an expert golfer.

George Alfred Nelson was elected a Member of the American Society of Civil Engineers on April 4th, 1911.

BENJAMIN FRANKLIN MORSE, M. Am. Soc. C. E.*

Died February 17th, 1914.

Benjamin Franklin Morse was born on June 7th, 1829, in South Kirtland, Geauga County, now Lake County, Ohio, and in 1836 became a resident of Painesville, in the same county. During his residence at Painesville, he attended the common schools and afterward the Painesville Academy; later, he studied mathematics and civil engineering with General E. A. Paine, a graduate of West Point, who had retired from the United States Army.

Mr. Morse was identified with much of the pioneer railroad building in Northern Ohio, in the capacity of Assistant Engineer, in which connection he represented the Lake Shore Railroad, aiding in the construction of the line between Cleveland and Erie, Pa., in 1851 and 1852.

In 1853, he was Assistant Engineer, under Major Potter, at the harbors of Fairport, Ashtabula, and Conneaut, Ohio. During the season of 1854 he was First Assistant, under Captain Howard Stansbury, U. S. Engineer, in the examination of the harbors on Lake Erie west of Cleveland, namely Lorain, Vermilion, Huron, Sandusky, and Monroe, Mich.

* Memoir prepared by J. F. Morse, Esq.

As First Assistant Engineer, Mr. Morse had charge of a line from Tiffin, Ohio, to Fort Wayne, Ind., now a part of the Nickel Plate System. During 1855 he was Assistant Engineer in charge of a survey for The Cleveland and Mahoning Railroad, from Youngstown, Ohio, to New Castle, Pa., and from 1857 until 1862, he was First Assistant, under Mr. Charles Collins, in the Engineering Department of the Lake Shore Railroad, between Cleveland and Erie, Pa.

In 1862 the four railroad companies, the Cleveland and Columbus, the Cleveland, Painesville, and Ashtabula, the Cleveland and Toledo, and the Cleveland and Pittsburg, proposed through their Presidents to build the present Union Station in Cleveland. The Presidents constituted the Building Committee, with Mr. Amasa Stone, of the Lake Shore, as Chairman, and he appointed Mr. Morse as his Engineer. The latter drew the plans for the station, and they were approved by Mr. Stone. They included what was probably at that time one of the largest train-sheds in the United States. Mr. Morse superintended the building of the Union Station, completing the work in 1865.

In 1868, as Chief Engineer, he surveyed the line from Chardon to Youngstown, Ohio, which is now a branch of the Baltimore and Ohio Railroad, extending from Fairport to Youngstown. As Chief Engineer, he also made a preliminary survey for a railroad from Cleveland to Sharon, Pa.

Mr. Morse afterward became interested in the erection of many of the public buildings in Cleveland and superintended their construction. He superintended the construction of the City Work House and drew plans and superintended the rebuilding of the Newburg State Hospital, which was destroyed by fire in 1872.

In April, 1875, Mr. Morse was appointed City Engineer of Cleveland, in which capacity he served for 9 years. He remodeled the plans and completed the Superior Street Viaduct, which was opened in 1878. While acting as City Engineer, he estimated and reported on several high-level bridges, and one of the plans he advocated, the Central Viaduct, was adopted and developed.

He also first suggested and planned for the intercepting sewer for the City of Cleveland. He was appointed by the Building Committee to superintend the construction of the Society for Savings Building, but before active work was begun, he engaged with the Lake Shore and Michigan Southern Railroad Company to look after the building and rebuilding of its stations at Toledo and Chicago.

In 1890, under the new Building Code, Mr. Morse was appointed Building Inspector, and served in that capacity for 3½ years. After retiring from this office he spent some time in travel, and laid aside active business except for occasional consultation work in engineering lines.

On September 19th, 1913, Mr. Morse was seriously injured in an automobile accident, from the effects of which he died on February 17th, 1914, at the Battle Creek Sanitarium where he had gone for treatment.

In 1855, Mr. Morse was united in marriage to Matilda Craft, of Tiffin, Ohio. He is survived by three sons and one daughter.

He was a Royal Arch Mason, and for many years belonged to the old Board of Trade, and also to the Chamber of Commerce, of Cleveland.

Mr. Morse was one of the oldest members of the American Society of Civil Engineers, having been elected a Member on July 12th, 1877. He was a Charter Member of the Civil Engineers Club of Cleveland, now the Cleveland Engineering Society.

PHILIP CHAPIN DAVIS, Assoc. M. Am. Soc. C. E.*

Died March 26th, 1914.

Philip Chapin Davis was born on September 14th, 1881, at Kalamazoo, Mich., in which city he obtained his early education. In 1906, he was graduated from the Engineering Department of the University of Michigan, with the degree of Bachelor of Science in Mechanical Engineering. In September of that year, he entered the employ of the Thompson-Starrett Company, General Builders, and was with that company on several notable engineering problems, both in Chicago and New York, until August, 1911, when he became associated with the Jobson-Gifford Company of New York, as Superintendent of Construction in connection with the electrification of the New York, New Haven and Hartford Railroad, between New York City and New Rochelle, and also between Stamford and New Haven.

On October 29th, 1908, Mr. Davis was married to Miss Bertha Shean, of Kalamazoo, Mich., who, with two sons, survives him. Mr. Davis died at the Neurological Institute, New York City, on March 26th, 1914, following a year's illness from recurring attacks of pernicious anæmia.

Mr. Davis was a member of Anchor Lodge, No. 87, F. and A. M., and a member of the Sigma Chi Fraternity. He was a young man of exceptional ability, and his friends and associates feel deeply their loss on account of his untimely death.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on June 4th, 1913.

* Memoir prepared by Erle K. Knight, Assoc. M. Am. Soc. C. E.

ROGER TIFFT HOLLOWAY, Assoc. M. Am. Soc. C. E.*

Died March 12th, 1914.

Roger Tifft Holloway, the second son of Henry F. and Metta J. Holloway, was born on August 29th, 1885, in Columbus, Ohio. In 1887, the family moved to Montclair, N. J., and he was educated at the public schools of that place. He was graduated from the High School in 1904, and was President of his class during his Junior and Senior years. In the fall of that year, he entered Cornell University, from which he was graduated in 1908, with the degree of Civil Engineer. While at Cornell, Mr. Holloway was elected a member of the Alpha Delta Phi and of the Sphinx Head, a Senior society, and he also belonged to the Cornell Glee Club.

In August, 1908, Mr. Holloway was engaged by the Turner Construction Company, of New York City, as Assistant to the Superintendent on reinforced concrete work, remaining with the Company until January, 1909. From March to October, 1909, he was employed as Structural Detailer and Draftsman with the Hay Foundry and Iron Works, at Newark, N. J. In February, 1910, with the writer, he engaged in private practice, as a Consulting Structural Engineer, under the firm name of Mead and Holloway, continuing as a member of the firm for two years. During this time he prepared plans, estimates, etc., for numerous alterations and contracts, among which were plans and specifications for the steelwork for the Irvington School, the Nurses' Home in connection with the Presbyterian Hospital, in Chicago, Ill., the Dock Street Pier, Philadelphia, Pa., etc. He also acted in an advisory capacity on this work.

In June, 1912, Mr. Holloway severed his connection with the firm of Mead and Holloway to engage in the general practice of civil engineering. He was also the New York representative of the London firm of Bagley, Mills and Company, and until his sudden death which occurred at his home in Montclair on March 12th, 1914, from septic poisoning, following an attack of tonsilitis, he was engaged in preparing plans, specifications, estimates, etc., for a number of constructions and alterations in and around New York City.

Mr. Holloway was always courteous, and, in his short professional career, had established a reputation for painstaking care and accuracy which won him many friends. The aptitude shown by him in his profession was probably inherited, his father being a member of the American Society of Mechanical Engineers, and a great-uncle, Mr. J. F. Holloway, having served a term as President of that Society.

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* Memoir prepared by Charles A. Mead, M. Am. Soc. C. E., supplemented by information on file at the Society House.

He was a member of the University Glee Club of New York, the Harlequins, of which he was President, the Montclair Athletic Club, the Alpha Delta Phi Club of New York, and Montclair Lodge No. 144, F. and A. M.

Mr. Holloway was prominent in the social activities of Montclair where his loss will be keenly felt. His engagement had recently been announced, and the wedding had been planned for the coming Autumn. Besides his parents, he is survived by two brothers and two sisters.

Mr. Holloway was elected a Junior of the American Society of Civil Engineers on May 31st, 1910, and an Associate Member on May 7th, 1913.

JOHANNES CORNELIS VLIEGENTHART, Assoc. M. Am. Soc. C. E.*

Died October 29th, 1913.

Johannes Cornelis Vliegenthart was born in Delft, Holland, on July 15th, 1876. In 1893, he entered the Polytechnical School of Delft as a student of engineering, and was graduated therefrom in July, 1899, with the degree of Civil Engineer.

In September, 1899, he entered the Government service of The Netherlands as Assistant Engineer in the Royal Corps of Waterstaat, in charge of river improvement works of the waterway from the Port of Rotterdam to the sea, at Hook of Holland and Hansweert. This work included dredging, constructing groins for regulating the channel, etc.

In November, 1901, Mr. Vliegenthart was appointed Chief Engineer of the Haiho River Conservancy Commission, with headquarters at Tientsin, China. This Commission was appointed by the Peace Protocol after the Boxer Insurrection of 1900, for the purpose of dredging and regulating the Haiho River, with a view to making it navigable from Tientsin to the sea for steamers drawing from 10 to 14 ft. of water. Under his direction three channels were constructed of a total length of 4 miles, thereby shortening the river by 15 miles and eliminating ten sharp bends. In addition to this work, Mr. Vliegenthart had charge of a survey of part of the Province of Chili, which included the cities of Peking, Tientsin, and Paoting. This survey was made chiefly for the purpose of studying the courses of the various tributaries of the Haiho River, looking to the improvement of such tributaries. He had promised to read a paper before the Koninklijk Instituut van Ingenieurs (Royal Institution of Netherland Engineers) on the great engineering works in North China in which he had had such a prominent part, but the fulfillment of this promise was prevented by his death.

* Memoir prepared by the Secretary from information on file at the Society House.

With the exception of a vacation in 1906, he remained in China until early in 1913, when he resigned his position with the Haiho Conservancy Commission and returned to his home in Holland. In September, 1913, on the election of the well-known engineer, Mr. R. R. L. de Muralt as a Member of Parliament, Mr. Vliegenthart was appointed to succeed him as Engineer of the "Waterschap Schouwen" (Hydraulic Company, Isle of Schouwen, Province of Zeeland). He remained in this position until his death which occurred on October 29th, 1913, at Zierikzee, Holland.

He belonged to that group of Netherlanders who have raised the reputation of Dutch engineers in foreign countries to a high standard, and his name will be held in honored remembrance as one of the pioneers who co-operated in the development of China.

Mr. Vliegenthart was elected an Associate Member of the American Society of Civil Engineers on June 5th, 1907.

PAPERS IN THIS NUMBER

"SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS." CLARENCE T. JOHNSTON. (To be presented Sept. 2d, 1914.)

"THE CONSTRUCTION OF THE KLONDIKE PIPE LINE." W. W. EDWARDS. (To be presented Sept. 2d, 1914.)

"THE CONSTANT-ANGLE ARCH DAM." LARS R. JORGENSEN. (To be presented Sept. 16th, 1914.)

"SUBAQUEOUS HIGHWAY TUNNELS." GEORGE DUNCAN SNYDER. (To be presented Sept. 16th, 1914.)

PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

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Discussion .... Apr., May, Sept., "
"**Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors.**" John R. Nichols .... Apr., "
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"**Topographical Surveys Made by the American Section of the International Boundary Commission, United States and Mexico.**" W. W. Follett .... Oct., 1913
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Discussion. (Author's Closure.) .... Feb., Mar., May, 1914
"**Grouted Cut-Off for the Estacada Dam.**" Harold A. Rands .... Jan. "
Discussion .... Mar., Apr., May, "
"**The Diversion of Irrigating Water from Arizona Streams.**" A. L. Harris .. Jan., "
"**Steel Stresses in Flat Slabs.**" H. T. Eddy, Esq .... Jan., "
Discussion. (Author's Closure.) .... Mar., May, "
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"**An Investigation of Sand-Clay Mixtures for Road Surfacing.**" John C. Koch .... Feb., "
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"**Report on a Series of Tests on Concrete Columns Reinforced with a Spiral of Steel.**" Messrs. C. G. Wrentmore, Hugh Brodie, and C. O. Carey .. Feb., "
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"**The Determination of Safe Yield of Underground Reservoirs of the Closed-Basin Type.**" Charles H. Lee .... Apr., "
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<u>MISSING</u>

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YEAR 1914

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XL—No. 7

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OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XL—No. 7

SEPTEMBER, 1914

Edited by the Secretary, under the direction of the Committee on Publications.

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American Society of Civil Engineers

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

House of the Society—220 West Fifty-seventh Street, New York.

Telephone Number..................5913 Columbus.
Cable Address...................."Ceas, New York."

*Elected to fill the vacancy caused by the death of Emil Gerber, Director, on April 16th, 1914.

Vol. XL. SEPTEMBER, 1914. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

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MINUTES OF MEETINGS
OF THE SOCIETY

September 2d, 1914.—The meeting was called to order at 8.30 P. M.; Vice-President Gardner S. Williams in the chair; Chas. Warren Hunt, Secretary; and present, also, 64 members and 5 guests.

The minutes of the meetings of May 20th and of the Annual Convention were approved as printed in *Proceedings* for August, 1914.

A paper by Clarence T. Johnston, M. Am. Soc. C. E., entitled "Some Principles Relating to the Administration of Streams," was presented by the Secretary, who also read a communication on the subject from Herbert E. Bellamy, Assoc. M. Am. Soc. C. E.

A paper by W. W. Edwards, Assoc. M. Am. Soc. C. E., entitled "The Construction of the Klondike Pipe Line," was presented by the Secretary, who also presented communications on the subject from Messrs. G. B. Pillsbury and Walter S. Wheeler.

Messrs. J. H. Gandolfo, A. H. Van Cleve, and J. H. Granbery were appointed Tellers to canvass the ballot on the following proposed Code of Ethics:

"It shall be considered unprofessional and inconsistent with honorable and dignified bearing for any member of the American Society of Civil Engineers:

"1. To act for his clients in professional matters otherwise than as a faithful agent or trustee, or to accept any remuneration other than his stated charges for services rendered his clients.

"2. To attempt to injure falsely or maliciously, directly or indirectly, the professional reputation, prospects, or business, of another Engineer.

"3. To attempt to supplant another Engineer after definite steps have been taken toward his employment.

"4. To compete with another Engineer for employment on the basis of professional charges, by reducing his usual charges and in this manner attempting to underbid after being informed of the charges named by another.

"5. To review the work of another Engineer for the same client, except with the knowledge or consent of such Engineer, or unless the connection of such Engineer with the work has been terminated.

"6. To advertise in self-laudatory language, or in any other manner derogatory to the dignity of the Profession."

The Tellers reported as follows:

Total number of ballots received		2 316
Ballot in duplicate	1	
Ballots from deceased members	2	
Ballots in envelopes stamped, not signed	12	
Ballots in envelopes not signed, or otherwise irregular	19	
Ballots from members in arrears of dues	120	154
Ballots to be canvassed		2 162

Total affirmative votes	1 997
Negative votes	107
Blank	24
Modified	34
	2 162

J. H. Gandolfo,
A. H. Van Cleve,
J. H. Granbery.

The Chair declared the Code of Ethics adopted by the Society.

The Secretary announced the following deaths:

Benjamin Thomas Buffinton, of Fall River, Mass., elected Associate Member, September 7th, 1904; Member, January 3d, 1907; died July 6th, 1914.

Horace Crosby, of New Rochelle, N. Y., elected Member, February 17th, 1869; died July 24th, 1914.

Martin Luther Gardner, of New York City, elected Associate Member, September 2d, 1891; Member, March 4th, 1896; died May 14th, 1914.

Justus Herbert Grant, of Rochester, N. Y., elected Member, March 2d, 1892; died August 1st, 1914.

Hurd Clarence Hurd, of North Beach, Md., elected Member, July 2d, 1913; died July 8th, 1914.

Frank Parsons Lant, of New York City, elected Junior, October 3d, 1888; Member, February 1st, 1910; died June 30th, 1914.

Dickinson MacAllister, of Fort Hunter, Pa., elected Member, April 1st, 1896; died July 31st, 1914.

Henry Lewis Oestreich, of Brooklyn, N. Y., elected Junior, October 4th, 1892; Associate Member, December 6th, 1899; Member, April 6th, 1909; died August 13th, 1914.

George John Couchot, of San Francisco, Cal., elected Associate Member, May 3d, 1910; died May 3d, 1914.

Jesse Sidwell Matson, of Ashtabula, Ohio, elected Associate Member, July 1st, 1908; died July 4th, 1914.

John Marvin Peters, of McKeesport, Pa., elected Junior, May 1st, 1906; Associate Member, November 1st, 1910; died July 21st, 1914.

Philip Morris Pritchard, of Widnes, Lancashire, England, elected Associate Member, June 6th, 1900; died July 8th, 1914.

William de Hertburne Washington, of New York City, elected Associate Member, October 5th, 1892; died August 30th, 1914.

Warren Austin Gates, of Oklahoma, Okla., elected Junior, June 30th, 1911; died May 24th, 1914.

The Secretary announced the election of the following candidates on September 2d, 1914:

As Members

William George Addis, Winthrop, Mass.
Justin Kenderdine Anderson, Charleston, W. Va.
Charles Kennard Bowen, Los Angeles, Cal.
Henry Cyrille Breidert, Chicago, Ill.
Melville Fisk Clements, Ridgefield, Wash.
Edward Miall Durham, Jr., Atlanta, Ga.
Frank Henry Feller, Spokane, Wash.
Kirby Scott Heck, Manila, Philippine Islands
Alexander William MacCallum, Philadelphia, Pa.
John William Musham, Chicago, Ill.

Joseph Henry Prior, Chicago, Ill.
Charles Swift Riché, Galveston, Tex.
Otis Fletcher Rowland, Albany, N. Y.
Harry Edgar Sawtell, Winthrop, Mass.
Charles Henry Spencer, Washington, D. C.
Edward Gray Taber, Spokane, Wash.
William Chase Thomson, Montreal, Que., Canada

As Associate Members

Percy Lyons Abraham, Kingston, Jamaica
Langford Taylor Alden, San Francisco, Cal.
David Hawley Ashton, Salt Lake City, Utah
George Everett Baker, Whitehall, Mont.
Paul Leonard Bean, Orono, Me.
Walter Christmas Bodycomb, Winnipeg, Man., Canada
Archibald Alexander Brown, Oakland, Cal.
George Robert Ure Buchanan, Ensenada de Mora, Cuba
John Bow Challies, Ottawa, Ont., Canada
Charles Brown Cornell, Kent, Ohio
Ernest Buchanan Crane, Seattle, Wash.
Peter Francis Daly, North Bergen, N. J.
Edward Charles Davis, St. Louis, Mo.
Donald Derickson, New Orleans, La.
Rumley DeWitt, Troy, N. Y.
George Henry Ellis, Powell, Wyo.
Olney Norman Foote, Buffalo, N. Y.
Watson Gilbert Harmon, Detroit, Mich.
William Chaffin Howe, Worcester, Mass.
Leo Hudson, Pittsburgh, Pa.
Leonard Crouch Jordan, Brooklyn, N. Y.
Charles H Knowles, Manila, Philippine Islands
John Edward August Linders, Cleveland, Ohio
James Logan, Mt. Holly, N. J.
George Corpening Love, Saginaw, N. C.
Paul Wardlaw Mack, New York City
James Brownson McClain, Wilmington, N. C.
Gurdon Saltonstall Mumford, New York City
Edward Theobald Murphy, Boston, Mass.
Herbert Nunn, El Paso, Tex.
Herbert Osborne, Hope, B. C., Canada
Leon Friend Peck, Hartford, Conn.
Herbert Luther Pringle, Santa Marta, Colombia
John Burke Reddick, San Francisco, Cal.
Charles Herman Ruggles, West Palm Beach, Fla.
Daniel Rolland Schock, Potsdam, N. Y.

Alfred George Schutt, Detroit, Mich.
Samuel Osborne Scudder, West Philadelphia, Pa.
George Austin Sherron, Norwalk, Conn.
Raymond Sickles, Seneca Falls, N. Y.
Joseph Warren Silliman, Philadelphia, Pa.
Landon Garland Smith, Tupelo, Miss.
Arthur Vall Spinosa, Pittsburgh, Pa.
John Charles Keith Stuart, Montreal, Que., Canada
Douglas Barlow Turner, St. Louis, Mo.
Ephraim Martin Vail, Plainfield, N. J.
John Hynds Weidman, Syracuse, N. Y.
Ferdinand Jacob Frederick Weinert, Richmond Heights, Mo.
Roger Austin Wilson, Corozal, Canal Zone, Panama
Rollen Joe Windrow, Waco, Tex.
Francis German Wrightson, Jr., Sacramento, Cal.

As Associates

James Martin, Colfax, Cal.
Willard Adelbert Smith, Chicago, Ill.

As Juniors

Albert Mangum Alexander, Memphis, Tenn.
Ray Lester Allin, Pasadena, Cal.
Frank Louis Beal, Stamps, Ark.
Karl McCortle Cosgrove, Cambridge, Ohio
Benjamin John Curtis, Minneapolis, Minn.
Henry Hortensius George, 3d, Whitney, N. C.
Louis Richard Gons, Baltimore, Md.
Edward Crittenden Harding, Jr., Cincinnati, Ohio
George William Hawley, Escalon, Cal.
Maurice William Hewett, Malta, Mont.
Clarence Strain Jones, Leavenworth, Kans.
Joseph Oliver Kingsley, Santa Ysabel, Cal.
Carl Harman Knoettge, Denver, Colo.
Frank Kristal, Ann Arbor, Mich.
Harry Douglas Lovering, St. Paul, Minn.
Lloyd McEntire, Frenchtown, N. J.
Elbert Hume Reidpath, Buffalo, N. Y.
George William Richards, Harrisburg, Pa.
Harold Hersman Scott, St. Louis, Mo.
Charles Bach Seib, Hudson, N. Y.
John Zadok Street, Browning, Mont.
Edward Mayo Tolman, Baltimore, Md.

Maurice Anderson Webster, Philadelphia, Pa.
Charles Mallory Whelan, Portland, Ore.
Bernon Tisdale Woodle, Philadelphia, Pa.

The Secretary announced the transfer of the following candidates on September 2d, 1914:

From Associate Member to Member

Kay Alexander, Revelstoke, B. C., Canada
Alexander Conn Beeson, Washington, Pa.
Charles Davis Drew, London, E. C., England
Lars Rasmus Jorgensen, San Francisco, Cal.
Charles William McMeekin, Nevada City, Cal.
Frederick Mears, Seattle, Wash.
Robert Brooks Morse, Baltimore, Md.
Charles Walter Palmer, Philadelphia, Pa.
Harry Chittenden Vensano, San Francisco, Cal.

From Junior to Associate Member

Frederick Webber Amadon, New Haven, Conn.
George Erle Beggs, Princeton, N. J.
Sidney Raymond Bellows, New York City
Otto George Henry Buettner, New York City
Herbert Cram Ellis, White Plains, N. Y.
Horace Seth Griswold, Berkeley, Cal.
William Edward Hamilton, Cohoes, N. Y.
Richardson Hand, Wilkes-Barre, Pa.
Manton Hannah, Paris, Tex.
Daniel Hubbard, Baltimore, Md.
Howard Camberne Kirkwood, Flushing, N. Y.
Eugen Frederick Kriegsman, San Francisco, Cal.
Clarence Edward Long, Pittsburgh, Pa.
Donald Douglas Price, Lincoln, Nebr.
Arthur Richards, Kennett, Mo.
George Corliss See, Temple, Tex.
Hugo Conrad Soest, New York City
Louis Wachtel, Speculator, N. Y.
Russell Sherwood Wise, Passaic, N. J.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

September 2d, 1914.—The Board met at 3.30 P. M.; Vice-President Smith in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bates, Bush, Cummings, Edwards, Endicott, Loweth, Montfort, Ockerson, Thomson, Tuttle, and Williams.

The Constitution of the Baltimore Association of Members of the American Society of Civil Engineers was approved.

Five Members, 14 Associate Members, 2 Associates, and 5 Juniors, were dropped for non-payment of dues.

The resignations of 2 Associate Members and 2 Juniors were accepted.

Ballots for membership were canvassed, resulting in the election of 17 Members, 51 Associate Members, 2 Associates, and 25 Juniors, and the transfer of 19 Juniors to the grade of Associate Member.

Nine Associate Members were transferred to the grade of Member.

Applications for membership were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

October 7th, 1914.—8.30 P. M.—This will be a regular business meeting. Two papers will be presented for discussion, as follows: "External Corrosion of Cast-Iron Pipe", by Marshall R. Pugh, M. Am. Soc. C. E.; and "A Method of Determining Storm-Water Run-Off", by Charles B. Buerger, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for August, 1914.

October 21st, 1914.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "The Design and Construction of Four Reinforced Concrete Viaducts at Fort Worth, Texas", by S. W. Bowen, M. Am. Soc. C. E.; and "The Clarification of Sewage by Fine Screens", by Kenneth Allen, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for August, 1914.

November 4th, 1914.—8.30 P. M.—A regular business meeting will be held, and two papers will be presented for discussion, as follows: "Water Supply of the San Francisco-Oakland Metropolitan District", by H. T. Cory, M. Am. Soc. C. E.; and "The Lock 12 Development of the Alabama Power Company, Coosa River, Alabama", by E. L. Sayers and A. C. Polk, Members, Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendices* to the Annual Reports of the Board of Direction for the years ending December 31st, 1906, and December 31st, 1910, contain summaries of all searches made to date.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Friday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary of the Association, E. T. Thurston, Jr., M. Am. Soc. C. E., 713 Mechanics' Institute, 57 Post Street.

The by-laws of the Association provide for the extension of hospi-

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907); Vol. XXXVII, p. 28 (January, 1911).

tality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association

The meetings of the Colorado Association of Members of the American Society of Civil Engineers are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary, Roger W. Toll, Assoc. M. Am. Soc. C. E., 700 Tramway Building, Denver, Colo. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays, at 12.30 P. M., at the Colorado Electric Club.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association

The Atlanta Association of Members of the American Society of Civil Engineers was organized on March 14th, 1912. The Association holds its meetings at the University Club.

At the meeting of the Association on December 29th, 1913, the new Chairman, John Ruddle, M. Am. Soc. C. E., was installed, and Messrs. Park A. Dallis and G. R. Solomon were appointed members of the Executive Committee. T. P. Branch, Assoc. M. Am. Soc. C. E., was elected Secretary.

Baltimore Association

On May 6th, 1914, the Baltimore Association of Members of the American Society of Civil Engineers was organized, a Constitution adopted, and the following officers were elected: J. E. Greiner, President; Francis Lee Stuart, First Vice-President; L. H. Beach, Second Vice-President; Harry D. Williar, Jr., Secretary-Treasurer; and Messrs. H. D. Bush, B. T. Fendall, B. P. Harrison, Calvin W. Hendrick, Oscar F. Lackey, M. A. Long, and A. A. Thompson, Directors.

At its meeting of September 2d, 1914, the Board of Direction considered and approved the proposed Constitution of the Baltimore Association of Members of the American Society of Civil Engineers.

Louisiana Association

The Louisiana Association of Members of the American Society of Civil Engineers, has been organized with the following officers: Frank M. Kerr, President; J. F. Coleman and W. B. Gregory, Vice-Presidents; A. M. N. Blamphin, Treasurer; and L. C. Datz, Secretary.

Philadelphia Association

On December 22d, 1913, the Philadelphia Association of Members of the American Society of Civil Engineers was organized with the following officers: George S. Webster, President; Richard L. Humphrey and F. Herbert Snow, Vice-Presidents; John Sterling Deans, J. W. Ledoux, Edgar Marburg, and H. S. Smith, Directors; S. M. Swaab, Treasurer; and W. L. Stevenson, Secretary. The meetings of the Association will be held at the Engineers' Club of Philadelphia, 1317 Spruce Street.

Portland, Ore., Association

On June 18th, 1913, the Portland, Ore., Association of Members of the American Society of Civil Engineers was organized with the following officers: E. G. Hopson, President; W. S. Turner, First Vice-President; D. D. Clarke, Second Vice-President; G. B. Hegardt, Treasurer; and Charles J. McGonigle, Secretary.

Seattle Association

At the Annual Meeting of the Association, held on January 26th, 1914, the following officers were elected for the ensuing year: Ernest B. Hussey, President; A. H. Fuller, Vice-President; and Carl H. Reeves, Secretary-Treasurer.

Southern California Association

The Southern California Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary of the Association, W. K. Barnard, M. Am. Soc. C. E., 514 Central Building, Los Angeles, Cal.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association

At its meeting of March 4th, 1914, the Board of Direction considered and approved the proposed Constitution of the Spokane Association of Members of the American Society of Civil Engineers.

The following officers have been elected: President, C. S. MacCalla; Vice-President, U. B. Hough; Second Vice-President, Morton Macartney; Secretary-Treasurer, A. D. Butler.

Texas Association

At its meeting of December 31st, 1913, the Board of Direction considered and approved the proposed Constitution of the Texas Association of Members of the American Society of Civil Engineers.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 29 West Thirty-ninth Street, New York City.

Architekten-Verein zu Berlin, Wilhelmstrasse 92, Berlin W. 66, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 413 Dorchester Street, West, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 2511 Oliver Building, Pittsburgh, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, Room 6, City Bank and Trust Company Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Oregon Society of Civil Engineers, 605 Spalding Building, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE LIBRARY

(From July 31st to September 1st, 1914)

DONATIONS*

SYMMETRICAL MASONRY ARCHES,

Including Natural Stone, Plain-Concrete, and Reinforced-Concrete Arches, for the Use of Technical Schools, Engineers, and Computers in Designing Arches According to the Elastic Theory. By Malverd A. Howe, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9¼ x 6¼ in., illus., 24 + 245 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1914. $2.50. (Donated by the Author.)

The preface states that the object of this book is to present in simple form the method to be used in the design of masonry arches according to the elastic theory. In this, the second edition, the errors found in the first edition are stated to have been corrected and the demonstrations of the formulas to have been simplified. The greater portion of the book is devoted, it is said, to the solution of examples, each step being given in detail so as to be easily followed by the undergraduate or engineer. In general, the unit-load method has been used in solving the problems, as this, the author states, is the only satisfactory method to use if maximum stresses are desired. In Article 89 of Chapter IV, is given a method by which the effects of certain fields of loading can be found directly. In Table II of Appendix A, data are given for about 600 masonry arch bridges arranged by span. Appendix B contains a series of arch coefficients and an example illustrating their uses in transforming a given arch to one having different dimensions, and the latest knowledge concerning the maximum range of temperature in concrete arch rings is given in Appendix E. The Contents are: Fundamental Formulas; Symmetrical Arches Fixed at the Ends; Examples Showing the Application of the Formulas, etc.; Typical Arches; Appendices; Index.

INFLUENCE DIAGRAMS

For the Determination of Maximum Moments in Trusses and Beams. By Malverd A. Howe, M. Am. Soc. C. E. Cloth, 9¼ x 6 in., illus., 8 + 65 pp. New York, John Wiley & Sons, Inc.; London, Chapman & Hall, Limited, 1914. $1.25. (Donated by the Author.)

In this book, the author defines an influence diagram as "one which shows the effect of a unit load moving across a structure upon any function of the structure for any position of the load," and after giving a single, simple rule for drawing influence diagrams for bending moments for loads on ordinary trusses, he shows how that same rule may be applied for loads on continuous trusses, cantilever trusses, and arches. In addition, the author explains that while influence lines are usually constructed for the determination of maximum moments, they can also be easily drawn for stresses or even areas of truss members. Their use in the determination of criterions for the positions of wheel loads which produce maximums is also described and shown, it is stated, to be very simple. The Chapter headings are: Simple Trusses; Double Intersection Trusses; Continuous Trusses; Arches; Beams of Constant Cross-Section.

THE IRON ORES OF LAKE SUPERIOR

Containing Some Facts of Interest Relating to Mining and Shipping of the Ore and Location of Principal Mines; with Original Maps of the Ranges. By Crowell & Murray. Second Edition. Cloth, 9¼ x 6 in., illus., 257 + 5 pp. Cleveland, Ohio, The Pentor Publishing Company, 1914. $3.50.

Most of the subject-matter contained in this book, it is stated, is a compilation of information relating to the iron ores of the Lake Superior region which has appeared in the various trade journals, geological reports, and transactions of scientific societies. For this, the second edition, the authors have re-drawn all the maps and re-written much of the general matter. Descriptions of the geology

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* Unless otherwise specified, books in this list have been donated by the publishers.

and mineralogy of the region and the various iron ores are included, as well as of methods of mining, transportation, docks and dock machinery, and methods of sampling, concentrating and analyses of ores used in the region. In Chapter X, the authors have given, it is stated, the locations and latest descriptions of mines in the different ranges, together with names of operating companies, managers, sales agents, tables of yearly shipments and analyses of ores. The Index contains the name of each mine and ore referred to and the range to which it belongs. The Chapter headings are: The Early History of the Region; Geology; Mineralogy; Production of Ore; Dock Equipment; Classification of Ores; Beneficiation of Ores; Methods of Analyses; Fuel Engineering; Location and Description of Mines; Index.

BROWN'S DIRECTORY OF AMERICAN GAS COMPANIES:

Gas Statistics, 1914. Compiled and Corrected Annually by E. C. Brown. Twenty-seventh Annual Edition. Cloth, $10\frac{3}{4}$ x 7 in., 819 + 53 pp. New York, "The Gas Age", 1914. $5.00.

As stated in the title, this book contains gas statistics of many cities and towns in the United States, Canada, Mexico, the West Indies, and the Philippine Islands. The subject-matter is arranged alphabetically by States and cities, under the following classifications: Manufactured or artificial gas companies, natural gas companies, acetylene town plants, gasoline town plants, parent or operating companies, and an appendix which contains the financial data of the companies. Under each company is given the names of railroads and steamboat lines on which the place is located, as well as those of express companies delivering to that point, officers, process of manufacture, capacity of gas-holder, number of consumers, price, etc., of both the gas and electrical departments. The names and addresses of officers of the various public service commissions having jurisdiction over gas and electric companies are also given, as well as the officers, directors and committees of the different gas associations, together with an alphabetical list of members of such associations. There is also a company index and a city and town index, and a short list of books which have been found of value by gas engineers.

Gifts have also been received from the following:

Aldershot Gas, Water & Dist. Lighting Co. 1 pam.
Arizona-Corporation Comm. 1 bound vol.
Assoc. of Transportation and Car Accounting Officers. 1 vol.
Atlantic Deeper Waterways Assoc. 5 bound vol., 1 vol., 11 pam.
Augusta, Ga.-City Council. 1 pam.
Ayer, Fred. E. 1 pam.
Bellamy, Herbert E. 6 pam.
Bihar and Orissa, India-Public Works Dept. 1 vol.
Bombay, India-Public Works Dept. 1 bound vol.
Buffalo, Rochester & Pittsburgh Ry. Co. 1 pam.
Bureau of Ry. Economics. 9 pam.
Bush Terminal Co. 2 pam.
California-R. R. Comm. 1 bound vol.
California-State Board of Forestry. 2 pam.
California-State Board of Health. 1 pam.
California-State Reclamation Board. 1 pam.
California-Supt. of Public Instruction. 2 pam.
Canada-Dept. of Rys. and Canals. 2 vol., 10 maps.
Canadian Min. Inst. 1 pam.
Chicago, Ill.-City Waste Comm. 1 pam.
Clarksburg, W. Va.-Water Supply and Sewerage Board. 1 pam.
Connecticut-Public Utilities Comm. 1 pam.
Cosculleula, J. A. 1 pam.
Dooling, Peter J. 36 pam.
Fox, John A. 1 vol.
Franklin Coll. 1 pam.
Gloucester, Mass.-Board of Water Commrs. 1 pam.
Graf, Otto. 2 pam.
Great Britain-Mersey and Irwell Joint Comm. 1 pam.
Hall Signal Co. 1 bound vol., 1 pam.
Harrison, Fairfax. 1 pam.
Hartford, Conn.-City Clerk. 1 bound vol.
Horton, Robert E. 1 pam.
Idaho, Univ. of. 1 vol.
Illinois-Secy. of State. 1 pam.
Illinois-State Board of Equalization. 1 bound vol.
Illinois-State Geol. Survey. 1 pam.
Illinois-State Public Utilities Comm. 1 pam.
Illinois Steel Co. 1 pam.
Indiana-Public Service Comm. 2 pam.
Inst. of Industrial Research. 7 pam.
Inst. of Marine Engrs. 1 bound vol.
Institution of Engrs. of the River Plate. 1 pam.
Inter. Ry. Fuel Assoc. 1 vol.
Iowa-Executive Council. 1 pam.
Kansas-Public Utilities Comm. 1 pam.
Kansas-State Board of Health. 5 pam.
Lehigh Valley R. R. Co. 2 pam.
Louisiana-State Board of Equalization. 1 pam.
Louisiana State Univ. 1 vol.
Lynchburg, Va.-City Clerk. 1 vol.
McGee, Guy C. 1 bound vol.
Madison, Wis.-Board of Water Commrs. 1 pam.
Maine-State Water Storage Comm. 1 bound vol.
Maine, Univ. of. 1 vol., 1 pam.
Massachusetts-Board of Gas and Elec. Light Commrs. 1 vol.

Massachusetts-Highway Comm. 1 bound vol.
Massachusetts-Public Service Comm. 1 bound vol.
Mead, Elwood. 1 pam.
Melrose, Mass.-Supt. of Public Works. 1 pam.
Memphis, Tenn.-Artesian Water Dept. 1 pam.
Michigan Geol. and Biological Soc. 2 bound vol.
Minnesota-R. R. and Warehouse Comm. 1 bound vol.
Mississippi-Agri. Exper. Station. 2 pam.
Missouri-Bureau of Mines, Min., and Mine Inspection. 1 pam.
Missouri-Public Service Comm. 1 bound vol., 1 vol.
National Fire Protection Assoc. 1 pam.
Nelson, Knute. 2 pam.
New Hampshire-Public Service Comm. 1 pam.
New Jersey-Comptroller of the Treasury. 2 bound vol.
New South Wales-Board of Water Supply and Sewerage. 1 pam.
New York City-Board of Water Supply. 1 bound vol.
New York City-Comptroller. 1 pam.
New York State-Commrs. of the Palisades Interstate Park. 4 pam.
New York State-Dept. of Labor. 2 bound vol., 3 pam.
New York State-Public Service Comm., First Dist. 1 bound vol.
New York-State Engr. and Surv. 1 bound vol.
New York City Record. 2 vol.
New York Univ. 1 pam.
Newcastle-upon-Tyne, England-City Engr. 1 pam.
North Carolina-Geol. and Economic Survey. 1 pam.
O'Gorman, James A. 1 vol.
Ohio-Industrial Comm. 1 pam.
Ohio-State Fire Marshal. 1 pam.
Ohio State Univ. 11 pam.
Oklahoma-Geol. Survey. 1 pam.
Ontario, Canada-Dept. of Public Works. 1 bound vol.
Pennsylvania-Public Service Comm. 1 bound vol., 9 pam.
Pennsylvania-State Highway Dept. 1 pam.
Pennsylvania-Topographic and Geologic Survey. 1 bound vol., 1 pam., 1 map.
Permanent Inter. Assoc. of Road Congresses. 1 pam.
Portland, Me.-Commr. of Public Works. 1 pam.
Quebec, Canada-Dept. of Colonization, Mines, and Fisheries. 1 pam.
Rhode Island-State Harbor Improvement Comm. 3 pam.
Rosenthal, J. J. 1 pam.
Royal Soc. of Canada. 1 bound vol.
St. Louis, Mo.-Public Library. 1 pam.
Salem, Mass.-Water Dept. 1 pam.
Saskatchewan, Canada-Bureau of Public Health. 3 pam.
Smithsonian Institution. 3 pam.
Société de l'Industrie Minérale. 1 pam.
South Dakota-Board of R. R. Commrs. 1 bound vol.
Springfield, Mass.-Board of Water Commrs. 1 pam.
Switzerland-Abteilung für Landeshydrographie. 1 bound vol., 1 pam.
Tasmania-Geol. Survey. 2 pam.
Tennessee-Min. Dept. 1 pam.
Texas-R. R. Comm. 1 vol.
Tillson, George W. 1 pam.
Tribus, Louis L. 1 pam.
Tulane Univ. of Louisiana. 1 pam.
Union of South Africa-Rand Water Board. 1 pam.
U. S.-Bureau of Labor Statistics. 1 bound vol.
U. S.-Bureau of Mines. 5 pam.
U. S.-Bureau of the Census. 1 bound vol., 1 vol., 1 pam.
U. S.-Coast and Geodetic Survey. 2 vol., 2 pam., 2 maps.
U. S.-Geol. Survey. 6 vol., 17 pam.
U. S.-Gettysburg National Park Comm. 1 map.
U. S.-Interstate Commerce Comm. 2 pam.
U. S.-Reclamation Service. 1 pam.
U. S.-Secy. of the Interior. 2 bound vol., 5 pam.
Washington-Industrial Insurance Comm. 1 pam.
West Virginia-Board of Trade. 1 map.
West Virginia-Dept. of Mines. 1 bound vol.
West Virginia-Public Service Comm. 2 pam.
Wilgus, William J. 2 vol.
Wisconsin-Highway Comm. 1 pam.
Wisconsin-Tax Comm. 1 pam.
Wyoming-State Geologist. 7 pam.
Wyoming, Univ. of. 1 pam.

BY PURCHASE

Stages of the Ohio River and Its Principal Tributaries, 1858-89. Compiled by the United States Weather Bureau. Washington.

Interstate Commerce Commission Reports ; Vol. 24 : Decisions of the Interstate Commerce Commission of the United States, June, 1912, to October, 1912. Reported by the Commission. Wash., 1913.

Forschungsarbeiten auf dem Gebiete des Ingenieurwesens. Herausgegeben vom Verein Deutscher Ingenieure. Hefte 158, 159, 160. Berlin, 1914.

Grandes Voûtes. Par Paul Séjourné. Tome 1-4. Bourges, 1913.

Preventive Medicine and Hygiene. Milton J. Rosenau. New York, 1914.

Elemente des Kanalbaus. Von Theodor Heyd. Darmstadt.

Poor's Manual of Industrials, Manufacturing, Mining, and Miscellaneous Companies, 1914. New York.

Metallurgy of Copper. By H. O. Hofman. New York and London, 1914.

The Steel Foundry. By John Howe Hall. New York and London, 1914.

Lessons in Practical Electricity; Principles, Experiments, and Arithmetical Problems: An Elementary Text Book. By C. Walton Swoope. Fourteenth Edition, Revised and Enlarged, with Revision of Chapter on Electric Lighting by Harry Noyes Stillman. New York, 1913.

Modern Practice in Mining: Vol. 3, Methods of Working Coal. By R. A. S. Redmayne. New York and London, 1914.

SUMMARY OF ACCESSIONS

(From July 31st to September 1st, 1914)

Donations (including 22 duplicates)	316
By purchase	16
Total	332

MEMBERSHIP

(From August 7th to September 3d, 1914)

ADDITIONS

MEMBERS

		Date of Membership.
FRANCIS, HARRY VIVIAN. Care, P. C. Sewell, Yuille St., Brighton, Victoria, Australia		May 6, 1914
GRANT, JOHN ROBERT. Structural Engr., 601 Rogers Bldg., Vancouver, B. C., Canada.	Assoc. M. M.	Jan. 3, 1911 June 24, 1914
WULFF, EBERHARD JOHN. Pres., Wulff Eng. Co.; County Engr. and Supt. of Highways, Westchester County, Tarrytown, N. Y.		May 6, 1914

ASSOCIATE MEMBERS

DORRANCE, FRANK YOUNG. Prin. Asst. Engr., Montreal Filtration Works, 805 Durocher St., Montreal, Que., Canada	Jun. Assoc. M.	Feb. 5, 1907 June 24, 1914
ELLIS, EDWARD MYRON. Asst. Engr. in Chg., Barge Canal Contract 37, Minetto, N. Y.		May 6, 1914
HOPKINS, CHARLES ARCHER. Box 1511, Lewiston, Mont.		June 24, 1914
SAMPSON, FRANK WATKINS. 201 Texas Co. Bldg., Houston, Tex.		June 24, 1914
SCHENK, ERNEST EUGENE. 415 East 9th St., Waterloo, Iowa		June 24, 1914

JUNIORS

MILLS, GUY G. Care, Am. Bridge Co., Toledo, Ohio	June 24, 1914
WHITNEY, JOHN THAD. Care, U. S. Engr. Office, Dam 16, Ohio River, Bens Run, W. Va.	April 1, 1914

CHANGES OF ADDRESS

MEMBERS

ALLISON, WILLIAM FRANKLIN. 5264 Nineteenth Ave., N. E., Seattle, Wash.

BEACH, ROBERT JAMES. 103 Park Ave., New York City.

BONSTOW, THOMAS LACEY. Care, S. Pearson & Son, Ltd., Royal Albert Dock Extension, North Woolwich, London, E., England.

BURKE, MILO DARWIN. Cons. Engr., Room 706, Second National Bank Bldg., Cincinnati, Ohio.

BUSH, EDWARD WALLACE. 863 Asylum Ave., Hartford, Conn.

CARTY, JOHN EDWARD. Designing Engr., Bridge and Ferry Div., Public Works Dept., 602 City Hall Annex, Boston, Mass.

CHASE, FRANK LYNTON. Cons. Engr., Brunson Bldg., Columbus, Ohio.

DARNELL, JAMES LEE. Engr. and Contr., 335 Midland Bldg., Kansas City, Mo.

GRAY, HENRY LILBURN. Cons. Engr., 705 American Bank Bldg., Seattle. Wash.

HENOCH, MILTON JACOB. City Bldg. Insp., City Hall. Sioux City, Iowa.

MEMBERS (*Continued*)

MACKSEY, HENRY VINCENT. Care, Western Dominion Ry. Co., 61 Board of Trade Bldg., Calgary, Alberta, Canada.

MCCAUSLAND, CHARLES PATTERSON. Canton, Pa.

MCKIM, JAMES ARTHUR. Galveston, Tex.

PACKARD, RALPH GOODING. Pres., R. G. Packard Co., East 28th St., Bayonne, N. J.

PROUT, HENRY GOSLEE. 18 Stockton Pl., Nutley, N. J.

RICHARDSON, THOMAS FRANKLIN. 346 Montross Ave., Rutherford, N. J.

ROSS, DOUGLAS WILLIAM. Cons. Engr., 503 Charleston Bldg., San Francisco, Cal.

SAYERS, EDWARD LAWRENCE. Care, New River Co., Macdonald, W. Va.

STICKNEY, GEORGE FETTER. 1503 Farmers Bank Bldg., Pittsburgh, Pa.

WHITMER, DAVID HEIKES. Contr. Engr. (Whitmer Contr. Co.), P. O. Box 233, Pittsburg, Kans.

WILSON, FREDERICK CHARLES. 108 Blackman Ave., Joliet, Ill.

WOODS, ROBERT PATTERSON. Care, Board of Control, The Kansas City Rys. Co., Kansas City, Mo.

ASSOCIATE MEMBERS

BANDY, EDWARD LEE. Ada, Ohio.

BEE, CHARLES EVERETT. 806 East Seneca St., Ithaca, N. Y.

BERRY, FRANCIS RIGDON. 50 Broad St., New York City.

BRASLOW, BARNETT. Engr., Bureau of Municipal Research (Res., 668 East 220th St.), New York City.

BROWNELL, LEONARD DEMPSTER. Res. Engr., Barge Canal Terminals, 801 Chamber of Commerce, Buffalo, N. Y.

BURTON, WILLIAM. Const. Engr., Jause Bros., Boomer, Hughes & Crain, Calgary, Alberta, Canada.

COPLAND, ALEXANDER CHISHOLM. Asst. Engr., C. & O. Ry., Room 1804, C. & O. Ry. General Offices, Richmond, Va.

DOOLITTLE, FREDERICK WILLIAM. Director, Bureau of Fare Research, Am. Elec. Ry. Assoc., 29 West 39th St., New York City.

FOGG, PERCIVAL MORRIS. Engr., U. S. Reclamation Service, Lawton, Okla.

FORTER, SAMUEL ALEXANDER. Brigham City, Utah.

GARDNER, WARREN. With The Public Service Comm., First Dist., 207 West 56th St., New York City.

GEHRING, HERBERT AUGUST. Trinity College, Hartford, Conn.

GOODRICH, CLINTON RAYMOND. Care, James Stewart & Co., Inc., 300 North Broadway, St. Louis, Mo.

HANDEYSIDE, CHARLES AUGUSTUS. 1300 Old Colony Bldg., Chicago, Ill.

HARPS, HARRY MACY. 100 Lewis St., Perth Amboy, N. J.

JAMES, ALFRED RANDOLPH. Care, Speedwell Constr. Co., 119 West 40th St., New York City.

KALLASCH, WINFRED MILLER. Vice-Pres., Illinois Hydr. Stone & Constr. Co., Room 31, Hubbard Blk., Elgin, Ill.

KORSMO, AMUND MARIUS. 6226 Lakewood Ave., Chicago, Ill.

ASSOCIATE MEMBERS (*Continued*)

LIGHTFOOT, WILLIAM JOSEPH. U. S. Surv., Dept. of the Interior, Care, U. S. Surveyor-General, Dorris, Cal.

LONG, CLARENCE BURTON. Res. Engr., U. S. Reclamation Service, Fletcher, *via* Browning, Mont.

MACLEAN, WILLIAM EUSTACE. 911 Rogers Bldg., Vancouver, B. C., Canada.

MCINTYRE, WILLIAM AINSWORTH. Rd. Engr., Assoc. of Am. Portland Cement Mfrs., 808 Bellevue Court Bldg., Philadelphia, Pa.

MATSON, THOMAS HATCHER. Asst. Engr., U. S. Reclamation Service, 731 Wells Fargo Bldg., San Francisco, Cal.

MILLER, HIRAM. 307 Fountain St., Westville, Conn.

NEWTON, JOHN PARSONS. Hydr. Engr., Lake Erie and Ohio River Canal Board, 1503 Farmers Bank Bldg., Pittsburgh, Pa.

PERRY, FRANCIS WILLIAM. Engr. of Constr., Dept. of Bridges, New York City (Res., 161 Fenimore St., Brooklyn, N. Y.).

PETERSON, OTTO WALLACE. Fall River Mills, Cal.

PHILLIPS, SILAS BENT. Asst. Engr., Ore.-Wash. R. R. & Nav. Co., 1209 Wells Fargo Bldg., Portland, Ore.

RICE, JOHN MARIE THOMAS. Civ. Engr. with Morris Knowles, Cons. Engr., 6823 Thomas Boulevard, Pittsburgh, Pa.

ROBERTS, HARRY ASHTON. Asst. Supt., Montana Div., Ore. Short Line R. R., Lawrence, Kans.

SCHICK, HERMAN SCHOVE. Care, U. S. Engr.'s Office, Dam 17, Ohio River, Marietta, Ohio.

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JUNIORS

BANKS, CHARLES WILBUR. Asst. Engr., Board of Water Supply, 29 Mountain Ave., Mt. Kisco, N. Y.

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MORROW, CLARENCE EDGAR. Instr. in Architectural Eng., Mass. Inst. Tech., 1897 Beacon St., Brookline, Mass.

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EXPULSIONS

ASSOCIATE MEMBER	Date of Expulsion.
RIGHTMIRE, ESTEL DEAN	Sept. 2, 1914

DEATHS

GAY, LEON LINCOLN. Elected Junior, December 5th, 1905; Associate Member, October 2d, 1907; died May 6th, 1914.

GRANT, JUSTUS HERBERT. Elected Member, March 2d, 1892; died August 1st, 1914.

MACALLISTER, DICKINSON. Elected Member, April 1st, 1896; died July 31st, 1914.

OESTREICH, HENRY LEWIS. Elected Junior, October 4th, 1892; Associate Member, December 6th, 1899; Member, April 6th, 1909; died August 13th, 1914.

PETERS, JOHN MARVIN. Elected Junior, May 1st, 1906; Associate Member, November 1st, 1910; died July 21st, 1914.

WASHINGTON, WILLIAM DE HERTBURNE. Elected Associate Member, October 5th, 1892; died August 30th, 1914.

Total Membership of the Society, September 3d, 1914, 7 539.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(July 24th to August 29th, 1914)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal,* Assoc. Eng. Soc., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Institute Indicator,* Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *Engineering Record,* New York City, 10c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *Railway Age Gazette,* Mechanical Edition, New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Colliery Engineer,* Scranton, Pa., 25c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England, 6d.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 10c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Mining and Engineering World,* Chicago, Ill., 10c.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.

LIST OF ARTICLES

Bridges.

Cost of Labor and Materials Used in Painting the St. Louis Municipal Bridge.* R. D. Spradling. (86) July 15.
Bridge Pin-Boring Machine.* (14) July 22.
Large Masonry Arches on Swiss Mountain Railway.* J. C. Van Langendonck. (13) July 23.
Putting 13-In. Pins in Place of 10-In. Pins, Williamsburg Bridge.* O. E. Hovey. (13) July 23.
The Superstructure of the New Quebec Bridge.* (12) July 24.
High Cantilever Bridge in Mexico.* (23) July 24.
Ultimate Strength of Carbon-Steel Models of Quebec Bridge Members.* (14) July 25; (12) July 31.
Standard I-Beam and Pile Highway Bridges of the Iowa State Highway Commission.* (86) July 29.
The Swiss Government Bridge Rules of 1913. (23) July 31.

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Bridges—(Continued).

Approaches to North Side Point Bridge over Allegheny River at Pittsburgh, Consisting in Part of Reinforced Concrete Arch Spans and in Part of Solid Earth- and Slag-Fill between High Retaining Walls. (14) Aug. 1; (86) Aug. 26.
Heavy Loading Tests of Concrete Piles; Cast-in-Place Piles Driven in Filled Ground and Subjected to 36-Hour 60-Ton Loads with 3/16-Inch Average Settlement.* (14) Aug. 1.
Hamburg Turnpike Lift Bridge.* (14) Aug. 1.
A Concrete-Incased Steel Viaduct at Chicago.* (13) Aug. 6.
The Ohio River Bridge at Metropolis; Record-Breaking Simple Truss Span.* (13) Aug. 6.
Noteworthy Concrete Structures on the Lake Erie & Eastern R. R.* (13) Aug. 6.
Load-Tests on Concrete Piles, North Side Point Bridge Approach, Pittsburgh.* (13) Aug. 6.
Construction Plant Layout for a 1 200-Ft. Concrete Arch Bridge.* E. P. Knollman. (13) Aug. 6.
The Scherzer Bridge Over the Pamban Channel.* (12) Aug. 7.
Notes on the Protection of the Foundations of Chepstow Bridge Over the River Wye, in Ferro-Concrete.* E. S. Sinnott. (104) Serial beginning Aug. 7.
Quebec Bridge Girders and Wind Anchorage.* (14) Aug. 8.
Cantilever Span of the Bloomfield Viaduct, Pittsburgh.* (13) Aug. 13.
Reconstruction of the Queensboro Bridge.* (13) Aug. 13.
The Highland Railway and Cannon Street Accidents.* (12) Aug. 14.
Track Arrangement Adopted for Queensboro Bridge, New York City. (14) Aug. 15.
Great Northern Ry. Improvements at Seattle, Wash.* E. E. Adams. (13) Aug. 20.
A Swiss Reinforced Concrete Arch Bridge.* (96) Aug. 20.
Replacing a Swing Bridge with a Vertical Lift Structure.* (15) Aug. 21.
A Unique Method of Strengthening a Truss Bridge.* Wm. H. Warnecke. (15) Aug. 21.
Skeleton Abutments for Railroad Viaducts.* (14) Aug. 22.
Hell Gate Arch Skewbacks: Details of Sectional Cast-Steel Bearings for the 977½-Foot Span.* (14) Aug. 22.
Bridge Cost Record Used by the Illinois Highway Commission. (86) Aug. 26.
Chicago Bridge Work by Day Labor.* Harry J. McDargh. (13) Aug. 27.
Burrard Inlet Bridge, Vancouver.* (96) Aug. 27.
Linking India and Ceylon by Railway, Difficulties of Bridge Construction with Native Labor.* F. C. Coleman. (19) Aug. 29.
Schwingungen von Brücken.* Alfred Hawranek. (69) July.
Brücke über den Nymphenburger Kanale im Zuge der Ludwig-Ferdinand Strasse.* Bosch. (78) July 18.

Electrical.

The Use of Synchronous Condensers with Transmission Lines.* H. B. Dwight. (5) Vol. 27, Pt. 2.
The Engineering Problem of Electrification. A. H. Armstrong. (5) Vol. 27, Pt. 2.
An Analysis of the Benefits of Electric Drive in Cement Mills.* Thos. H. Arnold. (2) July.
Power Problem in the Lehigh District.* Hermann V. Schreiber. (2) July.
The Differential Equations of Long Distance Transmission. Geo. R. Dean. (73) Serial beginning July 17.
The Latest Type of Rennerfelt Furnace.* (26) July 17.
The Use of Electric Motors in Railway Shops. B. F. Kuhn. (Abstract of paper read before the Am. Master Mechanics' Assoc.) (47) July 17.
The Thermo-Relay, a New Protective Device.* B. B. Beckett. (111) July 25.
History of Electrical Transmission. G. O. Wilson. (111) Serial beginning July 25.
Hydroelectric Construction in Northern Mexico.* A. C. Hobble. (27) July 25.
An Electric Air-Hammer Drill.* (13) July 30.
Government *v.* Private Ownership. T. P. Sylvan. (Abstract of paper read before the Richmond Telephone Soc.) (73) July 31.
Recent Extensions at the Newport (Mon.) Generating Station.* (73) July 31; (26) Aug. 14.
Standardization Rules of the American Institute of Electrical Engineers. (42) Aug.
A Distribution System for Power Purposes.* F. D. Nims. (42) Aug.
The Effect of Delta and Star Connections Upon Transformer Wave Forms.* Leslie F. Curtis. (42) Aug.
Economy in the Operation of 55 000 Volt Insulators.* M. T. Crawford. (42) Aug.
Erecting a Steel Tower Transmission Line.* Alfred Still. (83) Aug. 1.
Municipal Street-Lighting Plant at Kalamazoo, Mich.* (27) Aug. 1.

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*Illustrated.

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The Storm Detector.* W. H. Lawrence. (From *General Electric Review.*) (19) Aug. 1.
Moving Loaded Electric-Light Poles.* C. E. Drayer. (13) Aug. 6.
The Rating of Switchgear Conductors. B. E. G. Mittell. (73) Aug. 7.
Electric-Service System at Evansville, Ind.* (27) Aug. 8.
Resonance Tests of Long Transmission Line. A. E. Kennelly and Harold Pender. (27) Aug. 8.
The Development of Electric Traction. John R. Hewett. (From *General Electric Review.*) (96) Aug. 13.
Recent Developments in Mercury Vapour Converters.* Maurice Leblanc, Jr. (Translated from *L'Electro.*) (73) Aug. 14.
Application of Telephone Relays to Commercial Circuits. C. Robinson and R. M Chamney. (73) Aug. 14.
Concrete Pipe for Subaqueous Tunnels.* J. C. Lathrop. (13) Aug. 15.
Electric Distribution Standards in San Diego.* L. M. Klauber. (111) Serial beginning Aug. 15.
Single Unit 5 000-kw. Steam Plant at Alton, Ill.* (27) Aug. 15; (17) Aug. 15.
Determination of Wave-Length in Radiotelegraphy.* A. S. Blatterman. (27) Aug. 15.
Development of the Westinghouse Turbine Reduction Gear.* (19) Aug. 15.
Difficulties in Initial Operation of a 110 000-Volt Transmission Line.* Frank G. Allen. (13) Aug. 20.
Texas' Largest Central Station.* (27) Aug. 22.
Hydroelectric Station on the Cuyahoga River.* (27) Aug. 22.
Power Plant at Hillsboro, Ill.* Thomas Wilson. (64) Aug. 25.
The Helfenstein Large Electric Furnace.* C. Van Langendonck. (20) Aug. 27.
Design of Pole Foundations.* C. L. Christensen. (14) Aug. 29.
A Permanent Electric Current Without Electro-Motive Force.* (From *Nieuwe Rotterdam'sche Courant.*) (46) Aug. 29.
High-Tension Distribution in Central Texas.* (27) Serial beginning Aug. 29.
A New Marconi Transatlantic Service.* John L. Hogan, Jr. (27) Aug. 29.
Elektrische Beheizung von Gebäuderäumen. H. Tschirner. (7) July 4.
Ueber ein neues radiotelephonisches System.* Ludwig Kühn. (41) July 16.
Bemerkungen über das Silbervoltameter. W. Jaeger und H. von Steinwehr. (41) July 16.
Ein Beitrag zur Berechnung der Zahninduktionen in Dynamoankern.* F. Blanc. (41) July 23.
Die Hochspannungstechnik auf der schweizerischen Landesausstellung in Bern. Karl Kuhlmann. (41) July 23.
Betriebsregulierung von Leitungsnetzen durch Maximal-Zeitrelais. P. Bendmann. (41) July 23.
Das Audion als Generator für Hochfrequenzströme. Lee de Forest. (41) July 23.

Marine.

Twenty Years' Progress in Marine Construction. Alexander Gracie. (63) Vol. 194.
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Controlling Ships' Engines Directly from the Bridge.* (19) July 25.
The Armament of the Spanish Battleship *Espana.** (11) July 31.
The Mumford Paraffin Motors.* (12) July 31.
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Suction between Passing Ships.* Sidney A. Reeve. (11) Serial beginning Aug. 7.
How Yachts are Measured.* Herbert T. Wade. (46) Aug. 8.
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Mechanical.

The Use of Anti-Piping Thermit in Casting Steel Ingots.* E. A. Beck. (56) Vol. 45.
Commercial Production of Sound Steel Ingots.* Emil Gathmann. (56) Vol. 45.
Plant for Hadfield Method of Producing Sound Steel Ingots.* Robert A. Hadfield. (56) Vol. 45.
The Production of Solid Steel Ingots.* Benjamin Talbot. (56) Vol. 45.

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Mechanical—(Continued).

The Use of Steam Compression as a Means of Recovering Energy. Pietro Oppizzi. (From *Politecnico.*) (88) July.
An Analysis of the Benefits of Electric Drive in Cement Mills.* Thos. H. Arnold. (2) July.
The Application of Low-Temperature Carbonization to Gas-Works. G. E. Foxwell. (66) July 14.
Gas Firing for Boilers.* Birkholz. (Paper read before the Westphalian Dist. Assoc. of German Engrs.) (22) July 17.
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Buck's Gear-Cutting Machine.* (47) July 24.
200-Ton Cutting-Out and Forming Press.* (11) July 24.
Steering Air Craft at Sea.* (46) July 25.
The Large Quantity Measurement of Artificial Gases in Distribution Systems.* J. C. Wilson. (Paper read before the Wisconsin Gas Assoc.) (24) July 27.
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The Charging of Two-Cycle Internal Combustion Engines. B. Hopkinson. (Paper read before the North-East Coast Institution of Shipbuilders and Engrs.) (12) July 31; (11) July 17; (47) July 31.
Surface Combustion. William A. Bone. (29) Serial beginning July 31.
Feedwater and its Treatment.* William Borrowman. (Paper read before the Scottish Federated Inst. of Min. Students.) (22) July 31.
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Arcwall Coal Cutters.* N. D. Levin. (45) Aug.
The Plant of the Wyandotte Portland Cement Company.* (67) Aug.
The Handling of Coal at the Head of the Great Lakes.* G. H. Hutchinson. (55) Aug.
Modern Coal-Boat Unloading.* J. F. Springer. (45) Aug.; (46) Aug. 15.
Minneapolis Flour Milling.* Chas. A. Lang. (55) Aug.
Producer Gas from Low-Grade Fuels.* R. H. Fernald. (3) Aug.
The Manufactured Gas Industry in America.* Gilbert Colville Shadwell. (62) Aug. 3.
Recent Improvements in Gas Manufacture. Alfred E. Forstall. (Paper read before the Soc. of Chemical Industry.) (24) Aug. 3.
What Gas Can Do for Cinema Theatres.* H. C. Widlake. (66) Aug. 4.
Gadsden Steam Power Plant, Gadsden, Ala.* Warren O. Rogers. (64) Aug. 4.
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Link-Belt Malleable Foundry at Indianapolis.* (20) Aug. 6.
The Hayden Fine Spindle Automatic Screw Machine.* (72) Aug. 6.
Some Tests on a Diesel Engine. Walter S. Burns. (Paper read before the Institution of Engrs. and Shipbuilders in Scotland.) (47) Aug. 7.
Oils and Lubrication. L. D. Allen. (64) Serial beginning Aug. 4.
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Knock-Off Pressures for Hydraulic Baling Press Pumps.* P. H. Parr. (12) Aug. 7.
High-Speed Bearings. Gerald Stoney. (Paper read before the North-East Coast Institution of Engrs. and Shipbuilders.) (11) Aug. 7.
The Diesel Engine.* (From the *Sibley Journal of Engineering.*) (19) Aug. 8.
Plant of Dime Savings Bank Building, Detroit.* Thomas Wilson. (64) Aug. 11.
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Thermal Phenomena in Carbonization. Herold Hollings and John W. Cobb. (Paper read before the Institution of Gas Engrs.) (83) Aug. 15; (24) Aug. 24.
Determination of Ammonia in Gas. J. D. Edwards. (From *Circular* No. 34, U. S. Bureau of Standards.) (83) Aug. 15.
Concrete Pipe for Subaqueous Tunnels.* J. C. Lathrop. (13) Aug. 15.
Motor-Driven Pumps at Beardstown, Illinois.* (14) Aug. 15.
Coal and the Chemistry of Its Carbonization. John Harger. (Paper read before the Soc. of Chemical Industry.) (24) Aug. 17.
Proper Treatment of Crucibles in the Modern Foundry.* H. W. Gillett. (From *Bulletin* 73, U. S. Bureau of Mines.) (62) Aug. 17.
The Manufacture of Porcelain for Electrical Purposes.* E. T. Montgomery. (76) Aug. 18.
A Rating Chart for Centrifugal Pumps.* L. J. Bradford. (13) Aug. 20.
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Hand-Fired Furnaces for Water-Tube Boilers. Osborn Monnett. (64) Serial beginning Aug. 25.
Important Steps in Manufacturing Immense Turbo-Generators.* (72) Aug. 27.
Traveling Car-Tipple on the Sag Canal.* (13) Aug. 27.
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Kalkbrennöfen mit Generatorgasfeuerung.* (80) July 9.

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*Illustrated.

Metallurgical.

Why Does Lag Increase with the Temperature from which Cooling Starts? Henry M. Howe. (56) Vol. 45.
The Gay-Lussac Method of Silver Determination. Frederic P. Dewey. (56) Vol. 45.
The Microstructure of Sintered Iron-Bearing Materials.* B. G. Klugh. (56) Vol. 45.
New Design of Open-Hearth Steel-Furnace Using Producer Gas.* Herbert F. Miller, Jr. (56) Vol. 45.
Notes on Cast-Iron.* Albert Sauveur. (56) Vol. 45.
The Hardinge Conical Mill.* H. W. Hardinge. (56) Vol. 45.
Notes on the Formation of Ferrites in Roasting Blende. G. S. Brooks. (56) Vol. 45.
The Role of Certain Metallic Minerals in Precipitating Silver and Gold. Chase Palmer and Edson S. Bastin. (56) Vol. 45.
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Two-Blast-Furnace Plant at West Duluth.* (14) July 25.
The National Zinc Company, Bartlesville.* E. H. Leslie. (103) July 25.
Theory and Practice of Sherardizing.* Samuel Trood. (20) Serial beginning July 30.
Research Work in the Laboratory and Mill. Warren F. Bleecker. (105) Aug.
Modern Research in the Metallurgy of Iron. Allerton S. Cushman. (3) Aug.
A New Blast Furnace to Smelt Concentrâtes.* Ellis W. Honeyman. (16) Aug. 1.
Bullion Melting as Practised at Smelters. Harold French. (83) Aug. 1.
Smelting Costs and Prices for Silver-Lead Ores. L. S. Austin. (103) Aug. 1.
Metallurgical Plants near Salt Lake City.* (16) Aug. 8.
Electrostatic Ore Separation.* I. C. Clark. (16) Aug. 8.
The Raritan Copper Refining Works at Perth Amboy, New Jersey. H. B. Pulsifer. (83) Aug. 8.
The Heat Treatment of Carbon Steel. H. P. Tiemann. (Paper read before the Technology Club of Syracuse.) (24) Aug. 10.
Collinsville Smelter of the Bartlesville Company.* E. H. Leslie. (103) Aug. 8.
Titration Results in Cyanidation.* A. W. Allen. (103) Aug. 15.
The Schoop Process of Metal Spraying.* (20) Aug. 20.
Amalgamation at Liberty Bell Mill. C. Lee Peck. (16) Aug. 22.
Ampere Efficiency in Electrolytic Refining.* Kenneth S. Guiterman. (16) Aug. 22.
Zinc Smelting at Hillsboro, Illinois.* E. H. Leslie. (103) Aug. 22.
Development of Gold Mining in the Philippines. C. M. Eye. (103) Aug. 22.
Nouvelles Recherches sur les Alliages de Cuivre et de Zinc.* Léon Guillet. (92) July.
Le Traitement des Minerais Aurifères du Chatelet (Creuse).* (33) July 25.
Der Riesendrehrohrofen und die Frage seiner Wirtschaftlichkeit. J. H. Schutt. (52) June 15.
Die neue Hochofenanlage der Vereinigten Hüttenwerke Burbach-Eich-Düdelingen in Esch a. d. A.* Hubert Hoff. (50) Serial beginning July 16.
Weitere Beobachtungen über die Zeilenstruktur, ihre Entstehung und ihre Beseitigung durch Wärmebehandlung. Paul Oberhoffer und Hans Meyer. (50) July 23.

Military.

The Aeroplane in War. W. S. Brancker. (19) Aug. 1.
Correctors for the Sighting Gear of Heavy Ordnance.* (11) Aug. 7.
The German Artillery in the War.* (46) Aug. 15.
Night Landing Signals for War Aeroplanes.* (46) Aug. 29.
L'Epreuve Militaire d'Endurance des Automobiles de Poids Lourds, du 1er au 30 juillet 1914.* Duroc. (33) Aug. 8.

Mining.

The India Mica Industry.* A. Faison Dixon. (56) Vol. 45.
A Problem in Mining, Together with Some Data on Tunnel-Driving.* F. M. Simonds and E. Z. Burns. (56) Vol. 45.
Electric Power Installation at El Tigre, Sonora, Mexico. James W. Malcolmson. (56) Vol. 45.
The London Mine, Mosquito Mining-District, Park County, Colo.* Charles J. Moore. (56) Vol. 45.
Valuation of Iron-Mines. James R. Finlay. (56) Vol. 45.
The Utilization of Exhaust-Steam for Collieries, Ironworks, etc., and the Cost of Electric Current Generated.* W. C. Mountain. (Paper read before the North of England Inst. of Min. and Mech. Engrs.) (106) Vol. 47, Pt. 3.
Allerton Bywater Collieries. H. F. Smithson. (Paper read before the Midland Inst. of Min., Civ., and Mech. Engrs. and the Midland Counties Institution of Engrs.) (106) Vol. 47, Pt. 3.
Ledstone Luck Colliery. J. G. Linneker. (Paper read before the Midland Inst. of Min., Civ., and Mech. Engrs. and the Midland Counties Institution of Engrs.) (106) Vol. 47, Pt. 3.

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Safety in Metalliferous Mines and Quarries. (Abstract of Report of the Royal Comm. on Metalliferous Mines and Quarries.) (68) July 11.
Hydraulic Mine Filling.* Charles Enzian. (From *Bulletin* 60, U. S. Bureau of Mines.) (57) Serial beginning July 17.
Barriers for Arresting Coal Dust Explosions.* George S. Rice and L. M. Jones. (From the *Black Diamond.*) (57) July 17.
Colliery Consumption and Machine Economy at an Upper Silesian Colliery. Karl Schultze. (Paper read before the Midland Inst. of Min., Civ., and Mech. Engrs.) (57) Serial beginning July 24; (22) July 24.
Switch-Gear for Mines.* E. Kilburn Scott. (Abstract of paper read before the Midland Inst. of Min., Civ., and Mech. Engrs.) (22) July 24.
Chiksan Mines, Chosen.* Clarence L. Larson. (16) Serial beginning July 25.
Use of the Vertical Sinking Pump.* E. Mackay Heriot. (16) July 25.
Mine Sampling at El Tigre.* Charles M. Heron. (16) July 25.
Arching in Collieries.* Robert G. Clark. (Paper read before the South Wales Inst. of Engrs.) (57) July 31.
The Britannia Colliery, Pengam.* (22) July 31.
Mine Roof Supports.* H. I. Smith. (45) Aug.
Maximum Coal Recovery. A. W. Hesse. (46) Aug.
Application of Electric Motors to Gold Dredges.* Girard B. Rosenblatt. (42) Aug.
Pembina Coal Co., Ltd.* E. I. Roberts. (45) Aug.
Steel Construction in Mining and Ore Reduction Plants.* C. A. Tupper. (82) Aug. 1.
Stoping Methods at the Golden Cross Mine.* Andrew W. Newberry. (16) Aug. 1.
Dredge Construction in Portuguese East Africa. Charles Janin. (103) Aug. 1.
Some Recent Improvements in Prussian Mining Practice.* (From *Zeitschrift für Berg, Hütten und Salinenwesen.*) (57) Aug. 7.
Utah Copper Co.* (16) Aug. 8.
A Hydraulic Sand-Mining Plant.* R. A. Boehringer. (13) Aug. 13.
The Faultless Faultfinder for Mining.* Walter Scott Weeks and Edward V. Huntington. (16) Aug. 15.
Moving Sand by Pumping.* (13) Aug. 20.
Operations of the Marysville Dredging Co.* W. H. Wright. (16) Aug. 22.

Miscellaneous.

Business Training for the Engineer. Alex. C. Humphreys. (8) July.
A Conference on the Relation between National and Local Engineering Societies, and Public Affairs. (98) July.
Review of Engineering Work in the State of Montana, 1913-1914. John H. Klepinger. (Paper read before the Montana Soc. of Engrs.) (1) July.
Stump Burning to Reclaim "Logged-Off" Lands; Methods and Costs of Clearing Tracts for Agriculture in Pacific Northwest.* Le Roy W. Allison. (14) July 25.
Graphic Methods of Presenting Data. Willard C. Brinton. (9) Serial beginning Aug.
The Engineer as a Factor in Modern Progress. Alex. C. Humphreys. (3) Aug.
Mortgagees of Machinery. George A. King. (14) Aug. 8.
Depreciation Accounting. Halford Erickson. (Paper read before the Wisconsin R. R. Comm.) (24) Aug. 10; (83) Aug. 1.
The Designing of Reinforced Concrete Retaining Walls by Comparison with the Determined Ratios of the Various Functions of the Height and Unit Pressures.* S. M. Cotten. (86) Aug. 12.
Further Studies of the Deodorizing Effects of Ozone. J. C. Olsen and W. H. Ulrich. (Abstract of paper read before the Am. Inst. of Chemical Engrs.) (13) Aug. 20.
Improvements in Land Clearing Machinery.* (18) Aug. 22.
The Nomenclature and Definition of Photometric Magnitudes and Units. A. P. Trotter. (Paper read before the Illuminating Eng. Soc.) (24) Aug. 24.
A Novel Circular Computer.* (13) Aug. 27.

Municipal.

The New Charter of the City of St. Louis. Wilbur B. Jones. (Paper read before the Engrs.' Club of St. Louis.) (1) July.
City Charters in General and the Proposed City Charter of St. Louis in Particular. Carl Gayler. (Paper read before the Engrs.' Club of St. Louis.) (1) July.
Control, Management and Maintenance of Rural Roads. J. Fred Hawkins. (Paper read before the Institution of Mun. and County Engrs.) (104) July 10.
The Economies of Modern Methods of Road Construction. Francis Wood. (Paper read before the Institution of Mun. and County Engrs.) (104) July 10.

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The Organization and Standards of the Iowa Highway Commission.* (86) July 15.
Dust Prevention Methods on District of Columbia Suburban Roads.* L. R. Grabill. (Paper read before the Am. Road Builders' Assoc.) (86) July 15.
Cost of Asphalt Paving Repair in St. Paul, Minn.* (86) July 15.
Grouting and Penetrating Methods of Road Surfaces. George Green. (Paper read before the Institution of Municipal and County Engrs.) (104) July 17; (96) Aug. 6.
Some Practical Points on Modern Road Work. W. H. Maxwell. (Paper read before the Institution of Mun. and County Engrs.) (104) July 17.
Recent Revisions in the Standards of the New York Highway Commission.* (86) July 22.
Bituminous Surfaces for County Roads: Permanence and Desirability.* A. B. Fletcher. (86) July 22.
Cleaning Old Paving Brick by Compressed-Air Hammers.* Charles S. Butts. (13) July 23.
Pavement Foundations for Heavy Traffic. Daniel B. Goodsell. (13) July 23.
Condition of Test Road after Two Years of Service; Report on Surface Treatments of Sections Built by Office of Public Roads at Chevy Chase, Maryland. (14) July 25.
Slag Paving Block in Washington and Baltimore. (14) July 25.
Industrial Track Equipment for Concrete Road Building. (14) July 25.
Construction and Maintenance of Sand Clay Roads in Georgia, Methods and Cost.* John C. Koch. (86) July 29.
Standard Cross-Sections for Illinois Roads.* (Abstract from Report of the Illinois Highway Comm.) (86) July 29.
Effect of High-Pressure Steam Mains under Streets.* (13) July 30.
The Early History of Bituminous Street Pavements.* (13) July 30.
The Use of Hydrated Lime in Concrete Mixtures for Pavements. Robert S. Edwards. (Abstract of paper read before the National Lime Mfrs. Assoc.) (67) Aug.
Bitulithic on Macadam on Marlboro Street, Boston, Mass.* (60) Aug.
Sub-Crust Movement in Roads Subject to Heavy Mechanical Traffic.* G. H. Jack. (104) Aug. 7.
The Perfection of the Modern Street Pavement.* Frank W. Cherrington. (60) Aug.
Railroad Contracting Methods Applied to the Construction of Highways; Organization Problem and Transportation of Material at a Minimum Expense Features of Central Ohio Highway Work.* (14) Aug. 1.
Methods and Cost of Road Maintenance in San Joaquin County, California.* W. B. Hogan. (86) Aug. 5.
A New Portable Asphalt Mixing Plant; Catskill Water Supply Roads.* (13) Aug. 6.
Moving a 40-Ton Rock without Injury to Pavements. (14) Aug. 8.
Connecticut's Concrete Roads.* (14) Aug. 8.
Methods of Sampling Materials of Construction Used by the New York Highway Commission. (86) Aug. 12.
Recent Developments in Granite-Block Paving.* (13) Aug. 13.
Road-Oil Specifications and Tests. Clarence B. Osborne. (From *California Highway Bulletin.*) (13) Aug. 13.
Applied Geology in Municipal Engineering. Herbert Lapworth. (Paper read before the Institution of Mun. and County Engrs.) (104) Aug. 14.
Concrete-Lined Highway Tunnel Carrying Heavy Loads from Railroads Above it.* (14) Aug. 15.
Organization and Standards of the Pennsylvania State Highway Department.* (86) Aug. 19.
Slag as a Road-Surfacing Metal. (96) Aug. 20.
The Results of City Planning in Essen, Germany.* Harold M. Lewis. (13) Aug. 20.
Rulings on Road Building in New Jersey. (14) Aug. 22.
Removing Core of Stockton Street Tunnel, San Francisco.* (14) Aug. 22.
Condition of Pavements in European Cities, Conclusions in Report of George W. Tillson. (14) Aug. 22.
Recent Paving Practice in Chicago.* A. J. Schafmayer. (14) Aug. 22.
Details of a Reinforced Concrete Pavement in Morgan Park, Ill.* (86) Aug. 26.
The Construction and Use of the Wooden Road Drag.* (86) Aug. 26.
The Relation of Farm Produce Hauling to Permanent Road Improvements.* (86) Aug. 26.
The Construction of Gravel Roads in Iowa.* T. R. Agg. (Abstract of Report to the Iowa Highway Comm.) (86) Aug. 26.
Amended Brick-Paving Specifications; Addition of Rattler-Test Requirement and Clause Requiring Lugs, and Prohibition of any but Batch Mixer, Feature New Rules. (13) Aug. 27.

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Municipal—(Continued).

Gasoline Tractors for Southern Road Work.* G. B. Buchanan. (13) Aug. 27.
London Asphalt Pavements Expensive; Averages for Rock Asphalt and Wood Block in Westminster and London Proper Show Latter to be 15 Per Cent. Cheaper. (13) Aug. 29.
Le Rôle de la Chaux mélangée au Béton des Chaussées. (84) July.

Railroads.

Piping and Segregation of Ingots of Steel and Ductility Tests for Open-Hearth Steel Rails. P. H. Dudley. (56) Vol. 45.
Comparative Notes on Steel-Rail Rolling. Robert W. Hunt. (56) Vol. 45.
Hardwood Pads for Railway Sleepers.* Krause. (From *Annalen für Gewerbe und Bauwesen.*) (88) July.
Notes on the Electrification of the French Midi Railway Company.* E. Vytborck. (88) July.
Single-Phase Traction and Weak Current Lines. G. Gironsse. (From *La Lumiere electrique.*) (88) July.
Brake Trials on Long Goods Trains. A. Huberti. (88) July.
Jubilee of the Buenos Ayres Great Southern Railway.* William Rögind. (23) July 10.
New East Coast Dining Car Trains.* (23) July 10.
Single-Phase Locomotives for the Rhaetian Railway.* (12) July 17.
The Amur Railway.* (23) July 17.
Locomotive for the P. L. M. Railway.* (11) July 17.
New Tank Locomotives for the East Indian Railway.* (23) July 17.
The Railways of the Sudan.* Percy F. Martin. (23) Serial beginning July 17.
Costs of Surfacing Track and Features Which Influence That Cost. (86) July 22.
Oscillating Signals for Grade Crossings.* (13) July 23.
The Hauenstein Base Tunnel, Switzerland.* Julian Grande. (12) July 24.
Great Indian Peninsula Railway Terminus Extension Scheme.* (23) July 24.
New Express Locomotives, South Eastern & Chatham Railway.* (23) July 24; (21) Aug.
New Low Grade Line from Tacoma, Wash., to Tenino.* (15) July 24.
Passenger Terminal Improvements at Buffalo.* (15) July 24.
Essentials in Successful Yard Operation. E. C. Tucker. (15) July 24.
Hump or Flat Yards. W. B. Hendricks. (15) July 24.
The Arnold Plan of Through Routes for Chicago Suburban Roads.* (18) July 25.
Combined Wood and Concrete Snowsheds in Cascades. (14) July 25.
Reconstruction of a Jersey City Railway Passenger Terminal.* (13) July 30.
Dump Cars for Transporting Garbage by Rail.* (13) July 30.
A Spring Guard-Rail for Frogs.* (13) July 30.
Railway Construction in Switzerland. S. Berg. (12) Serial beginning July 31.
The Montreal Terminal Branch of the Canadian Northern Railway.* (23) July 31.
Electrification of Heavy Mountain Grades.* Joseph P. Ripley. (23) July 31.
The Piercing of the Hauenstein.* (23) July 31.
A New Shunting Locomotive, Great Northern Railway.* (23) July 31.
The Federal Valuation of the S. P. L. A. & S. L. E. G. Tilton. (15) July 31.
Making Provision for Emergency Grain Cars.* W. J. Tollerton. (15) July 31.
New G. N. Line from Oroville to Wenatchee.* (15) July 31.
Will Government Regulation Succeed? Samuel A. Dunn. (Paper read before the Indianapolis Transportation Club.) (15) July 31.
The Railways of Russia. Edmond Thery. (15) July 31.
Draft Gear Performance (for Cars). E. S. Pearce. (25) Aug.
Engine House Efficiency. W. W. Smith. (Paper read before the General Foremen's Convention.) (25) Aug.
Electric Locomotive Data. F. D. Everett. (25) Aug.
Pennsylvania Steel Box Car.* (25) Aug.
Recent Locomotives, Paris-Lyons and Mediterranean Railway.* (21) Aug.
West Coast Joint Stock Sleeping Saloons.* (21) Aug.
Articulated Triplex Compound Locomotive, Erie Railroad.* (21) Aug.
Railway Signalling Exhibits at the Anglo-American Exhibition.* (21) Aug.
Central of Georgia Railway Co.* Albert M. Wolf. (87) Aug.
Bloomingdale Road Track Elevation Project, Chicago, C. & C. B. Division, C. M. & St. P. Ry.* Albert M. Wolf. (87) Aug.
New Engine Terminal, Lake Shore & Michigan Southern Ry., Air Line Junction, Ohio.* (18) Aug. 1.
Consolidated Locomotives for the Western Maryland Ry.* (18) Aug. 1.
Organization Methods for $10 000 000 Track-Elevation Program (Rock Island Railroad).* (14) Aug. 1.
Problem in Railway Curves.* F. C. Snow. (14) Aug. 1.
Recent American and European Express Locomotives.* (46) Aug. 1.
Electrification of Steam Railways. D. D. Ewing. (Paper read before the Indiana Eng. Soc.) (96) Aug. 6.

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Lining Long and Deeply Overlaid Tunnels.* Eugene Lauchli. (13) Aug. 6.
New Rolling Stock on the District Railway.* (23) Aug. 7.
New 4-6-0 Superheater Locomotives for the Ceylon Government Railway.* (23) Aug. 7.
The Saveoil Creosoting Machine (for Railway Sleepers).* (12) Aug. 7.
Decision in the Five Per Cent. Rate Advance Case. (15) Aug. 7; (18) Aug. 8.
Baltimore and Ohio 2-10-2 Type Locomotive.* (15) Aug. 7.
Santa Fe System Freight Loss and Damage Organization. H. R. Lake. (From papers read before the General Superintendents' Assoc.) (15) Aug. 7.
Lining Tunnels on the Grand Trunk Pacific.* (14) Aug. 8.
Continuous-Rail Crossing for Track Intersections.* (13) Aug. 13.
World's Largest Locomotive Tested for Tractive Effort.* (13) Aug. 13.
A Swinging Transfer Table for Shifting Locomotives.* (13) Aug. 13.
Railroad Development in the Philippine Islands. C. H. Farnham. (15) Aug. 14.
Building a Modern Terminal Road at Youngstown, Ohio.* (15) Aug. 14.
Pennsylvania Railroad X25 Steel Box Car.* (15) Aug. 14.
Time and Cost Saving with Movable Forms, Chicago, Rock Island & Pacific Reduces Time Element 75 Per Cent. and Cost 70 Per Cent. in Erecting Large Retaining Walls.* (14) Aug. 15.
Construction Plant for Mount Royal Tunnel.* W. C. Lancaster. (14) Aug. 15.
Concrete-Lined Highway Tunnel Carrying Heavy Loads from Railroads Above It.* (14) Aug. 15.
Rolling Stock for Montreal Tunnel and Terminal. W. C. Lancaster. (17) Aug. 15.
New Signals and Interlocking on the Long Island R. R. at Jamaica, L. I.* (18) Aug. 15.
The Old Cleveland Trainshed Roof.* (13) Aug. 20.
New Locomotive Terminal of the Central R. R. of New Jersey.* (13) Aug. 20.
Recent Railway Construction in Chile.* Charles P. King. (13) Aug. 20.
Fuel Economy on the Chicago, Burlington & Quincy. A. N. Willsie. (15) Aug. 21.
Making a Double Track Fill Nearly 185 ft. High.* (15) Aug. 21.
Raising and Shifting a Six-Track Main Line.* W. F. Rench. (15) Aug. 21.
Recent Tendencies Regarding the Canting of Rails.* (15) Aug. 21.
Possible Railway Routes through the Rockies. (14) Aug. 22.
Signaling and Interlocking of the Philadelphia Terminal, P. & R. Ry.* (18) Aug. 22.
An Interesting Method for Railway Rechaining.* (13) Aug. 27.
Calculating Earthwork Cross-sections.* (13) Aug. 27.
Rock Island Interlocking Plant at Joliet.* (15) Aug. 28.
Chicago, Burlington & Quincy 2-10-2 Freight Locomotive.* (15) Aug. 28.
New Shop Building Construction on Sunset Lines.* (15) Aug. 28.
Placing 300 000 Cubic Yards of Concrete, Special Plants and Extensive Use of Bulk Cement on Track Elevation of Chicago, Milwaukee & St. Paul Railway at Chicago.* (14) Aug. 29.
Les Chemins de Fer Agricoles de l'Egypte.* Edouard Harran. (38) May.
Note sur l'Usinage des Tiroirs Cylindriques de Distribution.* L. Fort. (38) July.
Nouvelle Locomotive à Tender Moteur des Chemins de Fer de l'Erié (E.-U.).* (33) July 25.
Projet de Voies Ferrées Monorails Suspendues Portant des Trains Electriques à Très Grande Vitesse.* A. Le Vergnier. (33) July 25.
Station Frigorifique Expérimentale, sur Wagon, de l'Association française du Froid.* Emile Gouault. (33) Aug. 1.
Bauanlagen für die Herstellung der elektrischen Zugförderung auf den Eisenbahnlinien Magdeburg-Bitterfeld-Leipzig-Halle.* Mentzel. (49) Serial beginning Pt. 7.
Neuartiger Verschiebebahnhof. K. Ruzsics. (102) July 15.
Kran für 30 t Last zum Heben von Tendern.* Bonnemann. (102) July 15.
Ueber die Stabilität von Tunnelmauerwerk; unter Berücksichtigung der Erfahrungen beim Bau des Hauenstein-Basistunnels.* E. Wiesmann. (107) July 18.
Der Schutz von Eisen-, Beton- und Verbundbauwerken über Eisenbahnbetriebsgleisen.* K. W. Schaechterle. (78) Serial beginning July 18.
Ueber die stereoskopische Messkunst und einen erstmaligen Versuch ihrer Anwendung bei Eisenbahnvorarbeiten in China.* Georg A. G. Müller. (40) Serial beginning July 22.
Vierachsiger Dynamometerwagen der Schweizerischen Bundesbahnen.* H. A. Gaudy. (107) Serial beginning July 25.
Bergschläge im Simplontunnel.* F. Rothpletz. (107) Aug. 1.
Amerikan Dampflokomotiven grosser Leistung.* H. A. Gaudy. (107) Aug. 15.

Railroads, Street.

Rail Feeders for Tramways. Henry M. Sayers. (73) July 17.
The Illumination of Tramcars. L. C. Porter and V. L. Staley. (Abstract of paper read before the Illuminating Soc.) (73) July 17.

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Tramway Experiences in Brazil. H. M. Sayers. (Paper read before the Newcastle-on-Tyne Congress of the Tramways and Light Railways Assoc.) (73) July 24.
Town Planning in Relation to Tramways. J. A. Brodie. (Paper read before the Newcastle-on-Tyne Congress of the Tramways and Light Railways Assoc.) (73) July 24; (104) Aug. 7.
Whitehall Street Tunnel, New York City; Twin Concrete-Lined Cast-Iron Tubes 6 500 Feet Long, with 300 Feet of Double-Track Concrete Arch at Manhattan End.* (14) July 25.
Open Cars Changed to Closed Center Entrance.* (17) July 25.
Tramway Rail Joints. Henry M. Sayers. (26) July 31.
Municipal Street Railway of Edmonton, Alberta, Can. (60) Aug.
New Rapid Transit Cars for London.* (17) Aug. 1.
Electric Railway at Constantinople.* (17) Aug. 1.
Rebuilding Summer Cars to Center Entrance Type.* G. M. Cameron. (17) Aug. 1.
Specifications Relating to Compressed Air in Tunnel Work. Frederick C. Noble. (From the *Compressed Air Magazine.*) (96) Aug. 6.
New Repair Shops at Louisville.* (17) Aug. 8.
Some Track Construction Standards for 1914.* (From *Electric Traction.*) (96) Aug. 13.
The Kansas City Street-Railway Franchise. (13) Aug. 13.
Illinois Traction System Special Service.* (17) Aug. 15.
Electric Safety Brakes.* (17) Aug. 15.
Manhattan Bridge Three-Cent Line.* (17) Aug. 22.
Old Slip Tunnel, New York City; Features of Section of New Subway System under East River, Connecting William Street, Manhattan, with Clark Street, Brooklyn.* (14) Aug. 22.
Subway Construction at Buenos Aires.* (14) Aug. 29.
Amerikanischer Strassenbahnoberbau. Günther. (39) Serial beginning July 5.

Sanitation.

Brief Review of Sewage Disposal Works in Some European Cities and Comparison with the Pennypack Creek Works at Philadelphia.* George E. Datesman, M. Am. Soc. C. E. (59) June.
Data and Discussion on the Design and Maintenance of Inverted Siphons in Sewers.* Frank H. Carter, Assoc. M. Am. Soc. C. E. (86) July 15.
A $190 000 Drainage Project in Kentucky.* (86) July 22.
Applied Geology in Municipal Engineering. Herbert Lapworth. (Paper read before the Institution of Mun. and County Engrs.) (96) July 23; (86) Aug. 19.
Notes on the Refuse Destructor Works and Electricity Undertaking of the Borough of Hackney. Leonard L. Robinson. (Paper read before the Institution of Mun. and County Engrs.) (104) July 24.
Using Plow to Open Pavements (Sewer Construction).* C. A. Bryan. (14) July 25.
Sewer Explosions and their Prevention. Leonard Metcalf and Harrison P. Eddy. (13) July 30.
Dump Cars for Transporting Garbage by Rail.* (13) July 30.
Tilbury and Some Particulars of a New Sewerage Scheme in the Adjoining Rural District.* S. A. Hill-Willis. (104) July 31.
Worcester Sewage Disposal Works.* T. Caink. (Paper read before the Institution of Mun. and County Engrs.) (104) July 31.
Automobile Street Cleaning Machines.* (60) Aug.
Garbage Incinerators and Destructors.* Sterling H. Bunnell. (60) Aug.
Imhoff-Tank Operation at Chambersburg.* C. F. and P. E. Mebus. (14) Aug. 1.
The Auburn Water Case, Sanitary Lawsuit Involving the Carrying of Pollution by Wind and Water Movements. (14) Aug. 1.
A Discussion of Relative Values in Sanitation. George C. Whipple. (Paper read before the Connecticut Soc. of Civ. Engrs.) (86) Aug. 5.
Shoring Difficulties in St. Louis Sewer Construction. Charles C. Phelps. (13) Aug. 6.
Washington Street-Cleaning, Field Methods.* J. W. Paxton. (13) Aug. 6.
Heating and Ventilation in a Theater.* (101) Aug. 7.
Outfall Sewer Construction, with Special Reference to Belfast. James Munce. (Paper read before the Royal Sanitary Inst.) (104) Serial beginning Aug. 7.
Sewage Disinfection in Actual Practice, Observations Based on the Operation of Twenty Plants in the State of New Jersey Using Calcium Hypochlorite as a Disinfectant. William J. Orchard. (14) Aug. 8.
Sanitation Work at Cuban Iron Mines.* Charles F. Rand. (103) Aug. 8.
Second Explosion in Trunk Sewer, Pittsburgh.* N. S. Sprague. (14) Aug. 8.
Explosions in Sewers; Tests as to the Explosive Qualities of Different Mixtures of Air and Gasoline Vapors Made at Yale University. (14) Aug. 8.
Electricity Versus Steam in Drainage Pumping.* (27) Aug. 8.
Heating the Largest Pier in the World.* Frank H. Jones. (64) Aug. 11.

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Method and Cost of Constructing an Inverted Sewer Siphon of 12-in. Cast Iron Pipe Encased in Concrete Beneath Small Stream at Carlisle, Pa.* C. A. Bryan. (86) Aug. 12; (13) Aug. 13.
Method and Cost of Making a Relocation Survey of Underground Pipe Lines, Cincinnati, Ohio.* O. E. Carr. (86) Aug. 12.
New Sewage Disposal Works at Bath.* (12) Aug. 14.
A Large Open-Air Swimming and Wading Pool.* (14) Aug. 20.
Surge Tank Problems. Franz Prasil. (96) Serial beginning Aug. 20.
Unsolved Problems of Forced Ventilation. C. E. A. Winslow. (Paper read before the Am. Inst. of Chemical Engrs.) (101) Aug. 21.
Public Comfort Stations in New York City.* (101) Aug. 21.
Economic Aspects of Intermittent Sand Filters. (14) Aug. 22.
Method and Cost of Making House Connections to the New Orleans Sewerage System. Rudolph Hering, George W. Fuller and Harrison P. Eddy. (Report to the Sewerage Board of New Orleans.) (86) Aug. 26.
The Probable Future of Various Sewage Treatment Methods. George W. Fuller. (Report to the Metropolitan Sewerage Comm.) (86) Aug. 26.
Construction Camp for Town of Torrance, Calif.* Ralph Bennett. (13) Aug. 27.
The Construction of a Small Sewerage System for Garfield, N. J.* Russell S. Wise. (13) Aug. 27.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XL. SEPTEMBER, 1914. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

WATER SUPPLY OF THE SAN FRANCISCO-OAKLAND METROPOLITAN DISTRICT.

By H. T. Cory, M. Am. Soc. C. E.

To be Presented November 4th, 1914.

Synopsis.

This paper deals particularly with the municipal water supplies, present and future, of the cities around the Bay of San Francisco, and incidentally of the so-called Hetch Hetchy project, for which Congress recently granted the necessary permits.

First, a detailed study is made of the population growth of the San Francisco-Oakland Metropolitan District and its several subdivisions. The conclusions reached are that, probably:

The East Bay portions will pass San Francisco proper in population, and to a greater extent in water requirements, in about 20 years;

The next census will show the entire district second to Greater Los Angeles; and that,

After a normal growth for two or three decades, Greater San Francisco-Oakland, and especially the East Bay district, will, for a few decades, grow very rapidly.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Then follows a brief description of existing systems and statements of their possible full development.

The question as to whether the Bay cities need and can afford a distant supply in the immediate future is then discussed.

Four or five decades hence, with the utmost co-operation of all interests, local supplies must be supplemented by water brought in from a distance.

The last two parts deal, respectively, with the considerations governing a wise choice of such distant supply or supplies, and the various possibilities.

The conclusions are that the problem is much more, and almost exclusively, one for the East Bay cities, than for San Francisco, and the desirability of considering the situation from this standpoint very carefully, before definitely fixing on the Hetch Hetchy project, is advised.

Introduction.

Urban communities on San Francisco Bay in California have proposed to secure additional water supplies. After more than a decade of endeavor, the City of San Francisco, acting in its own name, but latterly in behalf of a metropolitan water district, has just secured permission to use the Hetch Hetchy Valley and dam site on the main Tuolumne River, within the boundaries of the Yosemite National Park. The privilege of doing this was bitterly contested by numerous individuals, societies, and associations. Widespread interest was aroused, and much feeling engendered.

Because of the interesting nature of the problem to the Profession; because of the many engineering facts which have been brought out from time to time in a multitude of reports and hearings before Government officials and Congresses; and because various engineers have differed so widely in regard to many matters, a comprehensive paper on the subject is believed to be timely.

Topography of San Francisco Metropolitan Area.—San Francisco Bay, with its outlet to the ocean—the Golden Gate—is about in the middle of the California coast line.

The relief map, Fig. 1, shows the immediate region. The Northern Peninsula, generally referred to as Marin County, is quite hilly and

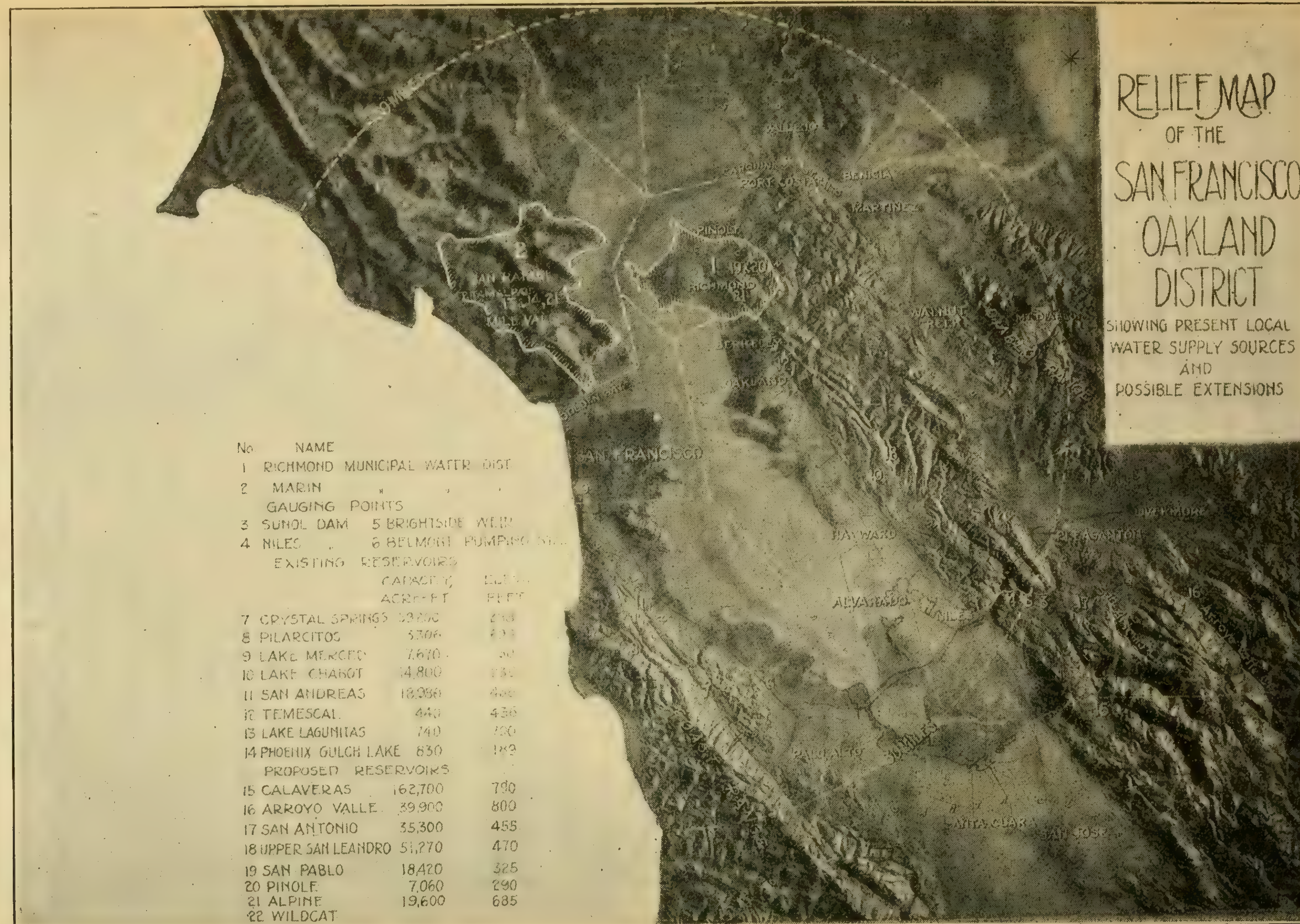
RELIEF MAP
OF THE
SAN FRANCISCO-
OAKLAND
DISTRICT
SHOWING PRESENT LOCAL
WATER SUPPLY SOURCES
AND
POSSIBLE EXTENSIONS
No NAME
1 RICHMOND MUNICIPAL WATER DIST
2 MARIN
GAUGING POINTS
3 SUNOL DAM 5 BRIGHTSIDE WEIR
4 NILES 6 BELMONT PUMPING
EXISTING RESERVOIRS
CAPACITY, ACRE-FT
FEET
7 CRYSTAL SPRINGS
8 PILARCITOS
9 LAKE MERCED 7,670
10 LAKE CHABOT 14,800
11 SAN ANDREAS 18,980
12 TEMESCAL 440 436
13 LAKE LAGUNITAS 740
14 PHOENIX GULCH LAKE 830
PROPOSED RESERVOIRS
15 CALAVERAS 162,700 790
16 ARROYO VALLE 39,900 800
17 SAN ANTONIO 35,300 455
18 UPPER SAN LEANDRO 51,270 470
19 SAN PABLO 18,420 325
20 PINOLE 7,060 290
21 ALPINE 19,600 685
22 WILDCAT
SAN FRANCISCO
OAKLAND
HAYWARD
ALVARADO
NILES
PLEASANTON
LIVERMORE
SANTA CLARA
SAN JOSE
BENICIA
MARTINEZ
PORT COSTA
PINOLE
RICHMOND

FIG. 1.

mountainous, with a relatively small area suitable for urban development. The Southern Peninsula, on which is the City of San Francisco, has a much more favorable, but still unfortunate, topography for city building. The portion south of the corporate line, and as far as the City of Santa Clara, is locally known as the "Peninsula," only the eastern edge of which is level or relatively so. The mainland along the east shore of the Bay from Richmond south has a comparatively narrow strip of level area, changing to rolling, and then to sharply hilly, country. This increases in width toward the south. The total area suitable for urban development on this side of the Bay is many times greater than that in San Francisco and both peninsulas combined.

The range of mountains running along the California coast line—the Coast Range—really ends in Twin Peaks, within the corporate limits of San Francisco, and begins again just across the Golden Gate, extending thence northward.

East of the Bay, another mountain range, parallel to the Coast Range but higher and wider, extends across the entire region, with a very narrow break, known as Carquinez Straits, through which the combined waters of the Sacramento and San Joaquin Rivers are discharged into San Francisco Bay. In many ways this Mt. Diablo Range is a most important topographical feature of the San Francisco Metropolitan District. It modifies in a striking way the local climatic conditions; stands as a barrier against urban growth eastward; affords productive water-sheds and storage facilities for local water developments; and is a very serious obstacle to be overcome in bringing a water supply to the Bay region from those sections of the Sierra Nevada Mountains directly to the east.

Urban Development.—The Mission of San Francisco was established by the Franciscan Fathers on October 9th, 1776, practically in the geographical center of the present City of San Francisco. In 1846 there was only a little settlement, called Yerba Buena, on the shores of Yerba Buena Cove, between Telegraph Hill and Market Street. The name was changed to San Francisco in 1847. With the discovery of gold in California in 1848, the settlement suddenly became important. For a few decades the growth was phenomenal and then dropped to about the normal in cities of its size.

It is unfortunate that the original settlement was not on the mainland, as the available area for satisfactory city development on the Southern Peninsula is relatively small, and is sharply restricted by the topography.

Effect of Topography.—The effect of the local topography has been very marked. The disposition of the land masses affects the fog distribution and results in a peculiarly great variation in climatic conditions over the district. San Francisco's climate is foggy and windy, generally speaking, with summer temperatures only slightly higher than those of winter, the thermometer rarely reaching 80° Fahr., and even a few scattering snowflakes and thin films of ice are to be seen only at intervals of years.*

The city is compactly built, due to the limited area which is not on steep hillsides. As the climate is really quite equable, it is unfortunate that home building was not carried out along the same lines as in Los Angeles and all other California cities, namely, to accentuate and take advantage of the famous climate.

In 1874, the area was materially enlarged by the introduction of cable cars, which were developed to meet the local situation. A railroad built into the interior of the State in September, 1869, naturally ended on the east shore of the Bay, with ferry service into the city proper. The suburban service thus available to the mainland, started settlements in Alameda, Oakland, and Berkeley, where ample space, milder climatic conditions, and cheap individual water supplies resulted in a different type of city building.

At first ocean shipping was of pre-eminent importance, and the development was determined by deep-water wharfage. As railway transportation increased in relative importance, commerce was influenced by the railway terminals on the mainland, but the position of San Francisco as the commercial center has become so firmly established that it is doubtful whether it will ever be transferred across the Bay.

Effect of Geographical Location.—In the nature of the case, San Francisco's population is cosmopolitan and self-sufficient. The city very quickly became a metropolis, and was effectually isolated from any other center. It was the financial and commercial hub of western gold mining enterprise, which attracted the energetic and adven-

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* The last snowfall was 1 in., on March 3d, 1896.

turous from all over the world. There are large foreign quarters in other cities of the country, but in San Francisco—except in Chinatown—the many nationalities are diffused. Consequently, its civic, religious, educational, and political ideas and institutions were and are different from those obtaining anywhere else in the United States. It is financially self-made, self-centered, and self-sufficient. The prevailing spirit was, and in large measure is yet, against San Francisco being a "cheap town." It is a wealthy one. Large personal freedom developed, and precedents of all kinds, including engineering methods, quickly came to be regarded lightly—often with excellent results.

In consequence, individualism was developed to an extreme, with a corresponding lack of co-operation. This condition obtains in a less, but still marked, degree to-day.

Considerable manufacturing necessarily developed, and, until the last decade, little was done on the Pacific Coast outside of the city. It became the center of Labor Unionism on the Coast, more particularly after the successful protests of Dennis Kearney and his followers against Asiatic labor competition.

Limits of Greater San Francisco-Oakland, as Used in This Paper.—The census adopts for metropolitan districts all urban population within 10 miles of the municipal boundaries. In this paper, however, the term will be used as including the territory which, logically, should ultimately have a common water supply. It is practically coincident with the region circumscribed by a circle of 30 miles radius with the San Francisco Ferry Building as the center. Greater Los Angeles, similarly, is considered as the area within a radius of 30 miles from the Los Angeles City Hall.

I.—Probable Future Population.

Growth of San Francisco.—For the past three decades the growth of the San Francisco Metropolitan District has been steady, and at the average rate for American cities of like population, but very much slower than Chicago, New York, and several others, when they were of similar size. The disastrous conflagration following the earthquake of April, 1906, temporarily affected San Francisco proper most seriously, but markedly accelerated the development of the East Bay cities and other suburbs. Such effect was not great enough to explain the relatively slow growth of Greater San Francisco-Oakland as com-

pared with all other important centers on the Pacific Coast and in the Far West during recent years.

The reasons for this, and how potent they will be in the future, are questions of primal importance. The population, and consequently the water requirements, are the first things to be determined in planning a water supply.

So far as the writer is aware, those who have estimated the future population of the district have not analyzed the peculiar conditions in California. The phenomenal percentages of increase in Los Angeles, Cal., Vancouver, B. C., Seattle, Wash., and other Pacific Coast cities in recent years, have never been taken into account, except by adding a statement that, remembering them, "the curve may be regarded as extremely conservative." This is not very illuminating.

Growth of California.—In the decade, 1900 to 1910, California's population increased 60.1 per cent. The rate for Oregon was 62.7%, Nevada, 93.4%, Idaho, 101.3%, and Washington, 120.4 per cent. The weighted average for these four other States was 95.9%, and for Washington and Oregon—the two other Pacific Coast States—95.8 per cent. The numerical increase was, Washington, 623 867; Oregon, 259 229; or together, 883 116—only 11 382 less than California.

Relative Growth of Northern and Southern California.—On examining detailed figures, it is found that the excessive population increase of the State is not of California generally, but of Southern California, consisting of the seven counties south of the Tehachapi—Ventura, Los Angeles, San Bernardino, Orange, Riverside, San Diego, and Imperial. The growth of population in these seven counties was 154%; in the rest of the State it was only 38%, and in the census San Francisco Metropolitan District but 41 per cent. The curves, Figs. 2 to 10, indicate that, unless something in the nature of widespread calamity or war occurs to change the respective rates of growth, Southern California will, by 1920, comprise more than half the State in all essential respects except area.

There are several reasons why this is surprising. They must in time be effective. These, in the probable order of importance, are:

1.—The natural resources of the State, like its precipitation, increase approximately in direct ratio from south to north. A marked

exception, whereby the reverse is true, is the attractiveness of the ocean beaches.

2.—The climatic conditions of Southern California, from a residence point of view, are inferior to those of the central and northern parts. This fact has been obscured by the effective advertising of Southern California. Greater San Francisco-Oakland is climatically

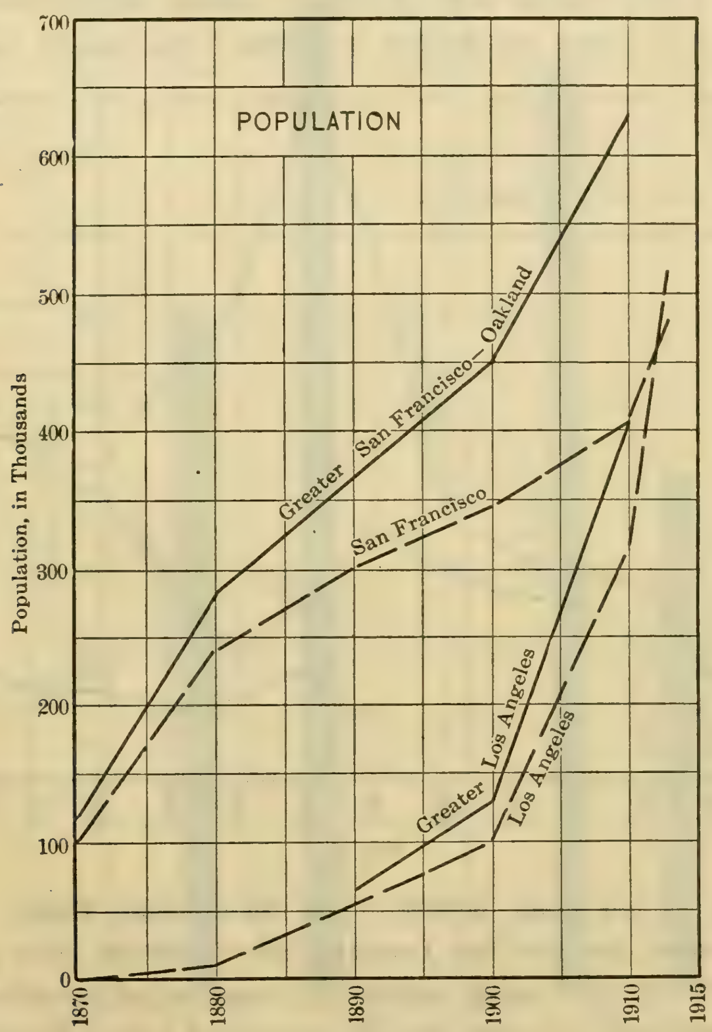


FIG. 2.

a markedly better summer resort than Los Angeles, and probably is not equalled anywhere in America.

3.—Within easy reach of San Francisco lies some of the most picturesque scenery in the world. There are opportunities to construct about the Bay cities the most attractive and beautiful roads

and boulevards on the Continent, and the Bay is almost unrivaled for yachting and motor boating the year round.

4.—The agricultural and horticultural possibilities of Southern California are literally dwarfed by those of the Central Valley—Sacramento and San Joaquin Valleys—alone. The total irrigated area of Southern California is a little more than 300 000 acres, exclusive of Imperial Valley (which will ultimately contain about 500 000 acres in the United States and 250 000 in Mexico), the present population

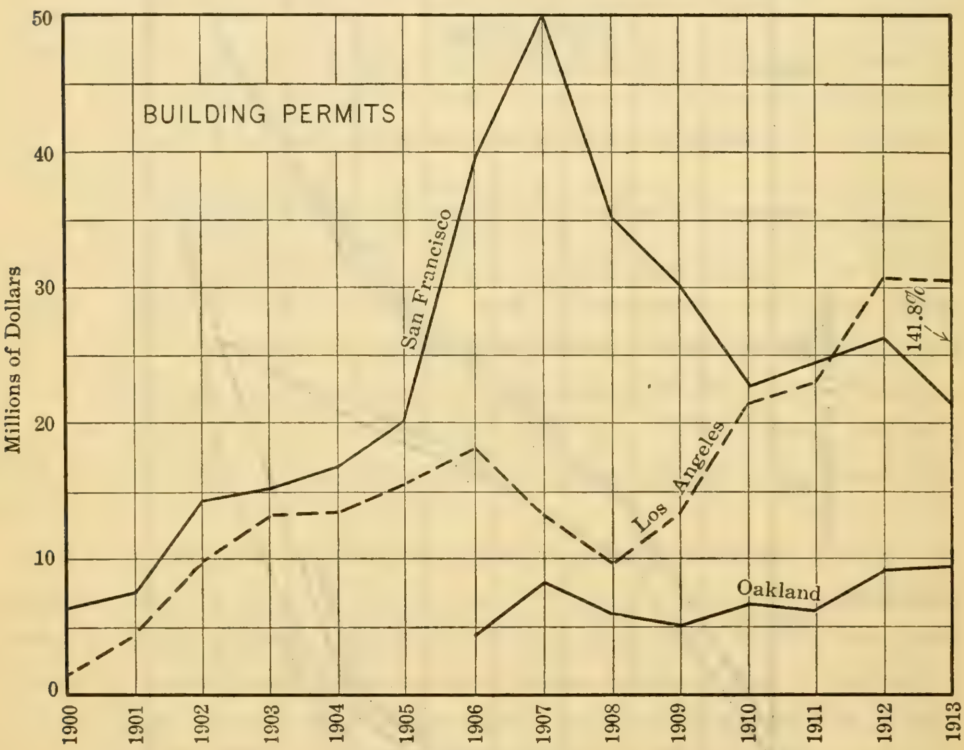


FIG. 3.

of which is less than 35 000. The Sacramento Valley contains 2 250 000 acres, and the San Joaquin Valley 5 000 000 acres of fertile land for which thoroughly satisfactory supplies of irrigation water may be developed at a cost per acre considered low in Southern California. That is, the agricultural development of all Southern California, which was the foundation for its entire development, is only 4% of the easily irrigable area of the Central Valley alone. Furthermore, the proportion of irrigable area adapted to growing oranges, lemons, grape-fruit, etc., is about the same in each region. When it is remembered that California produces 40% of the world's commerci-

ally grown oranges, and that the groves scattered about the Sacramento and San Joaquin Valleys have shown that oranges ripen there at least 6 weeks earlier than in Southern California, and suffer less from frosts, some realization of the situation can be obtained.

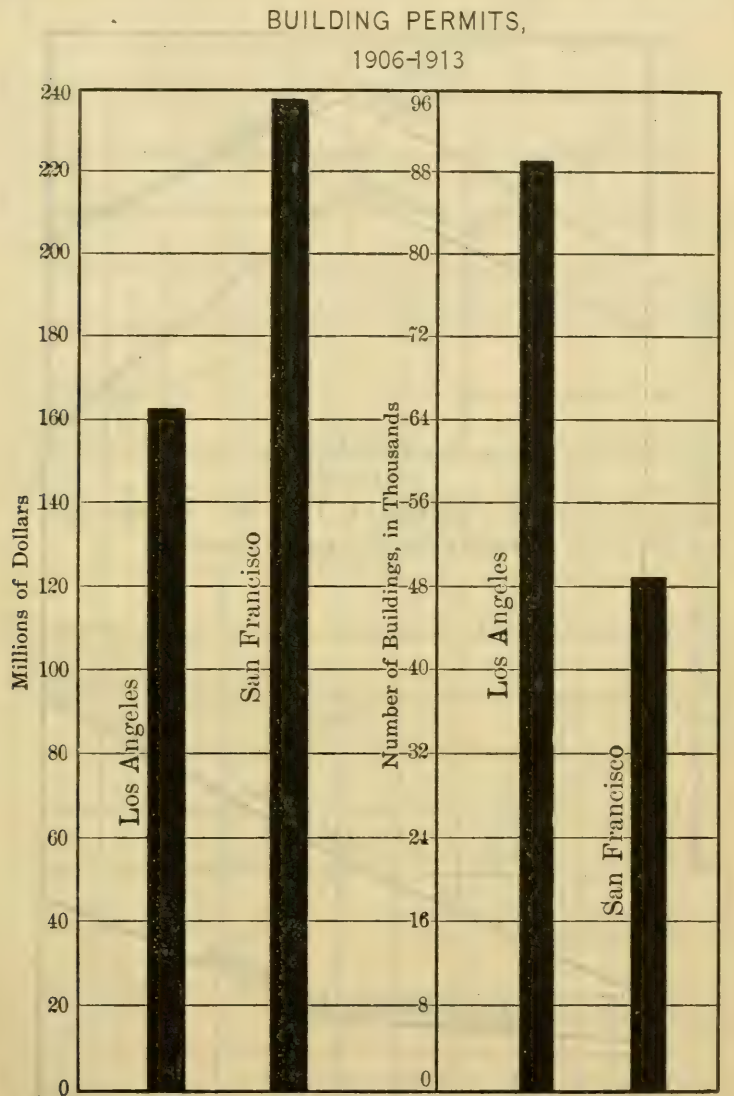


FIG. 4.

The total area of irrigable land logically tributary to San Francisco is 12 500 000 acres, and that of arable land is 15 393 000 acres.

5.—Thirty years ago, when Los Angeles had only 12 000 inhabitants—mostly Mexicans—San Francisco had 250 000, and was the "City" to which people "went down" from a vast region extending nearly 1 000 miles south, east, and north; it was practically the only

seaport on the Pacific Coast, and for more than 10 years was the western terminus of the only transcontinental railway.

In other words, the soil of Southern California is, to say the least, no more fertile; the water supply is by comparison pitifully

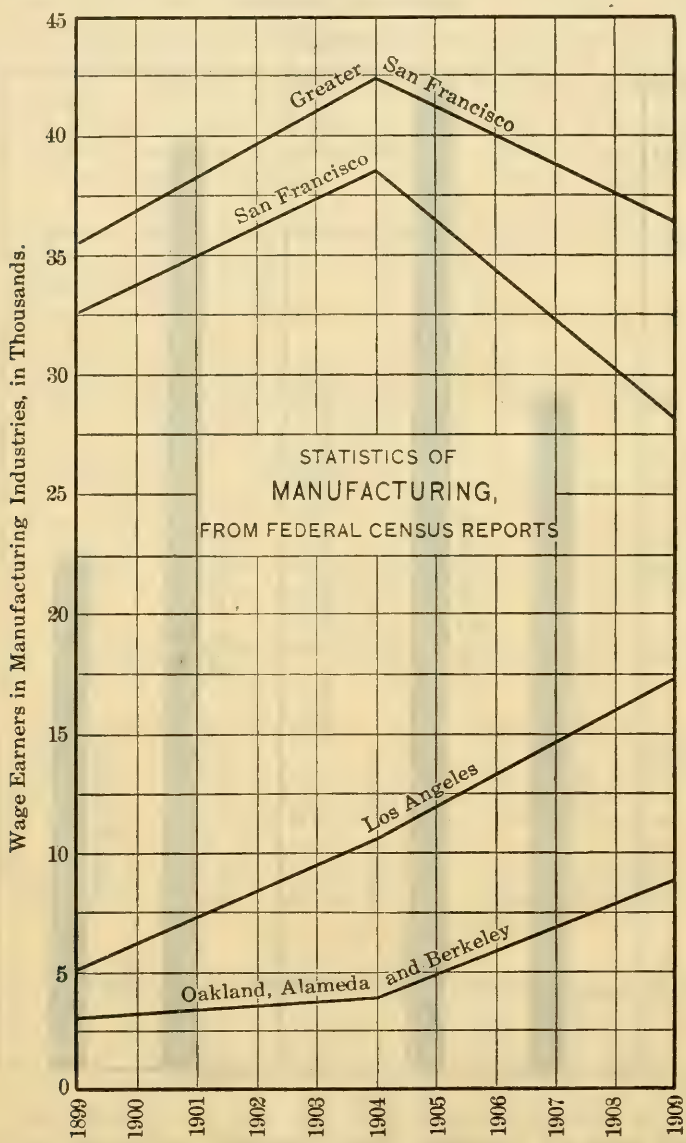
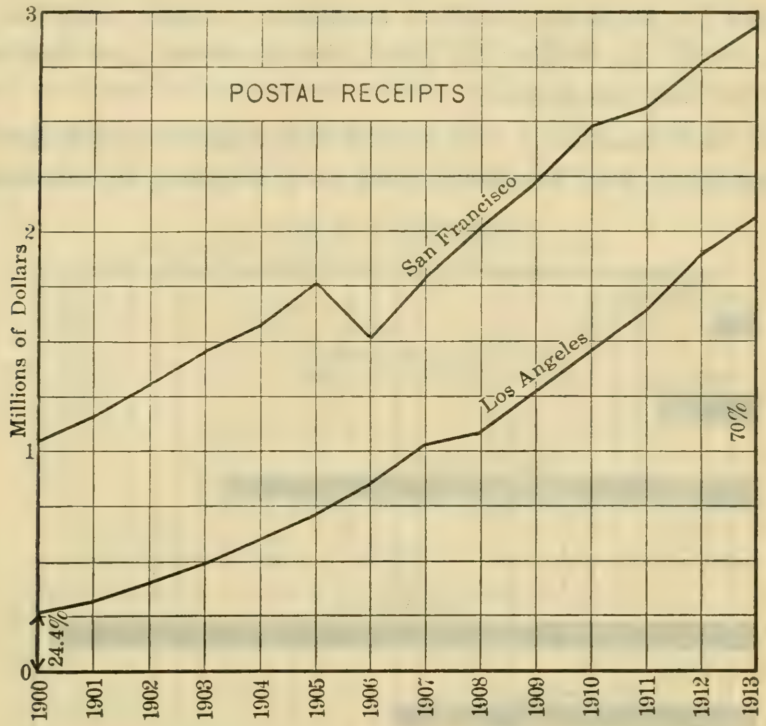


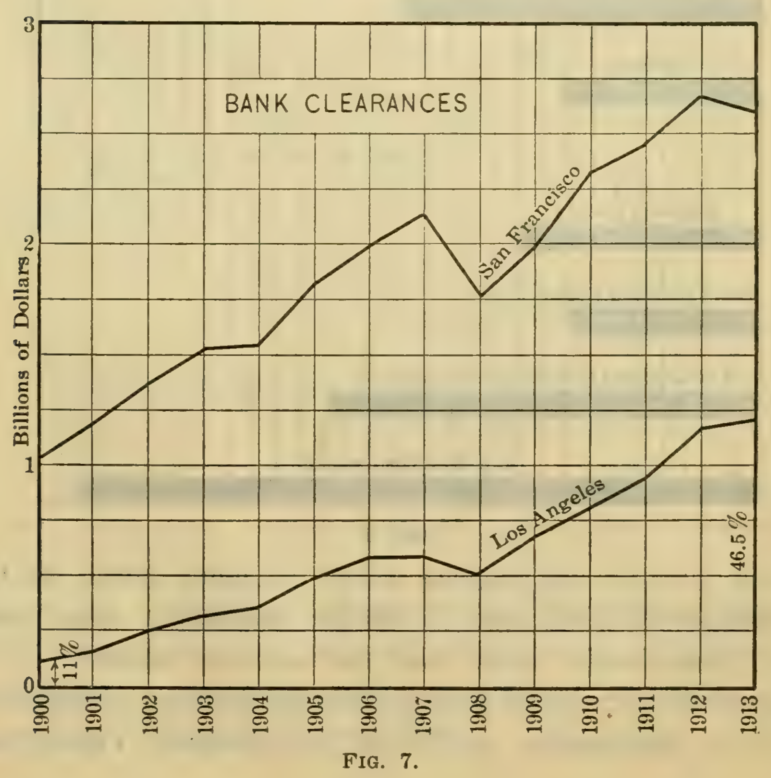
FIG. 5.

deficient; the climate not so delightful nor perfectly adapted to fruit culture, especially oranges and lemons; the extent of back country which can ever be made suitable for intensive farming, but a small fraction of that logically tributary to San Francisco; the natural



POSTAL RECEIPTS
Millions of Dollars
San Francisco
Los Angeles
70%
24.4%
0
1
2
3
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 6.



BANK CLEARANCES
Billions of Dollars
San Francisco
Los Angeles
46.5%
11%
0
1
2
3
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913

FIG. 7.

harbor and the shipping facilities hopelessly inferior; and the southern country so far behind in population 30 years ago that no one dreamed of real competition.

Under such conditions why should Los Angeles to-day have more inhabitants than San Francisco, with 86% as great an assessed valu-

STATISTICS FOR 1912.

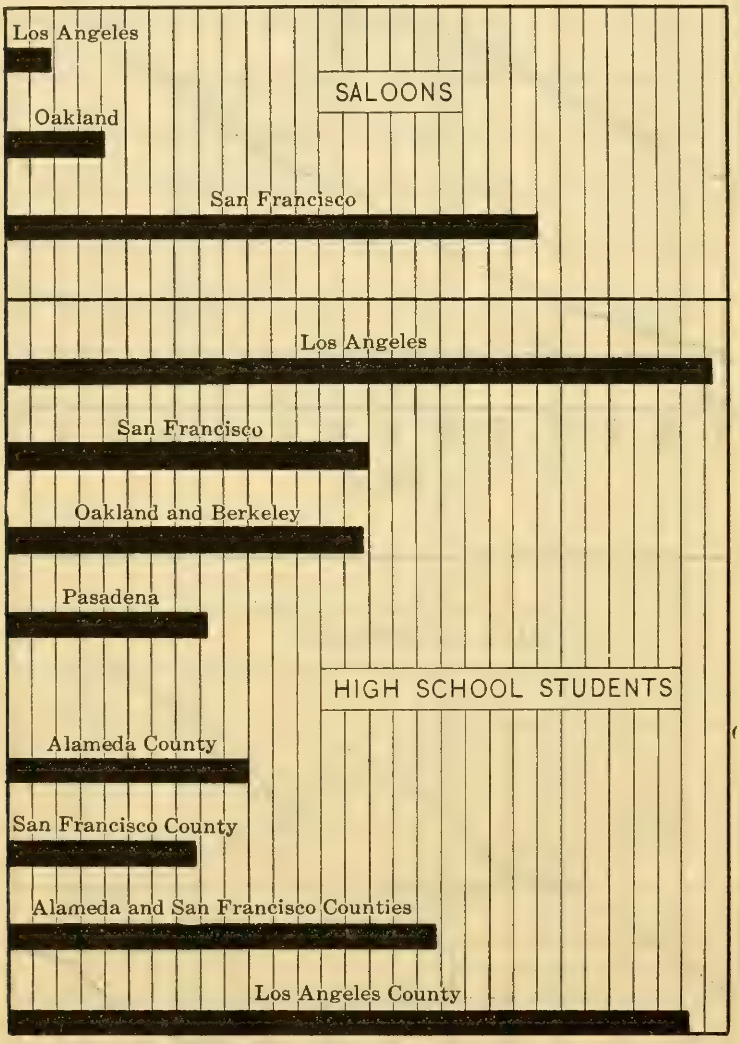


FIG. 8.

ation, and Greater Los Angeles nearly as many people as Greater San Francisco-Oakland, with a similar assessment ratio, and still growing tremendously faster than the northern center?

It seems to be a case of the hare and the tortoise. Natural advantages in the Sacramento and Upper San Joaquin Valleys resulted

in great bonanza grain farms—one man having raised on his own land holdings in a single season 1 000 000 bushels of wheat. It is an economic crime to devote such land, with its extraordinary crop adaptation and wonderful climate, to grain culture. In the Southland scanty rainfall compelled irrigation, and irrigation and citrus

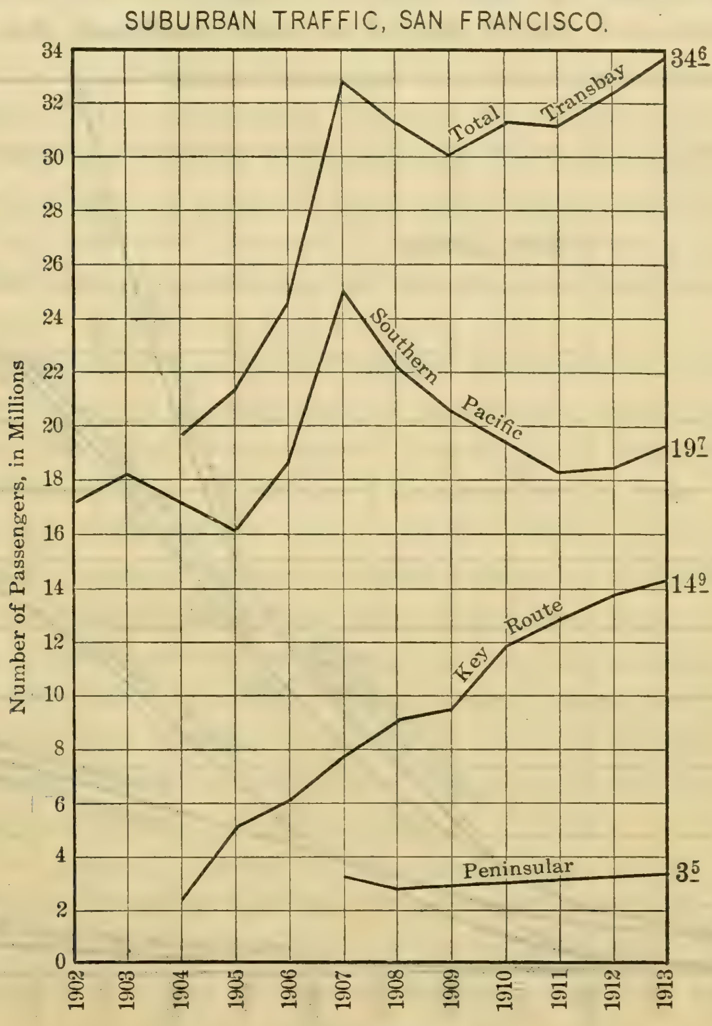


FIG. 9.

culture have been more highly developed there than anywhere else in the entire world.

However, the tremendous advantages of irrigation in the State were fully demonstrated 40 years ago, so that, in addition to this fundamental reason, there must be others quite as potent. These, in the probable order of importance, are:

1.—The Los Angeles Chamber of Commerce. This is doubtless the most efficient organization of its kind in the world, presenting a united, brave front in all matters affecting the development of, not only Los Angeles, but all Southern California. It is doubtless unequalled as an effective advertiser.

2.—The Pacific Electric Interurban System, for which Mr. H. E. Huntington, a former San Franciscan, furnished the financial

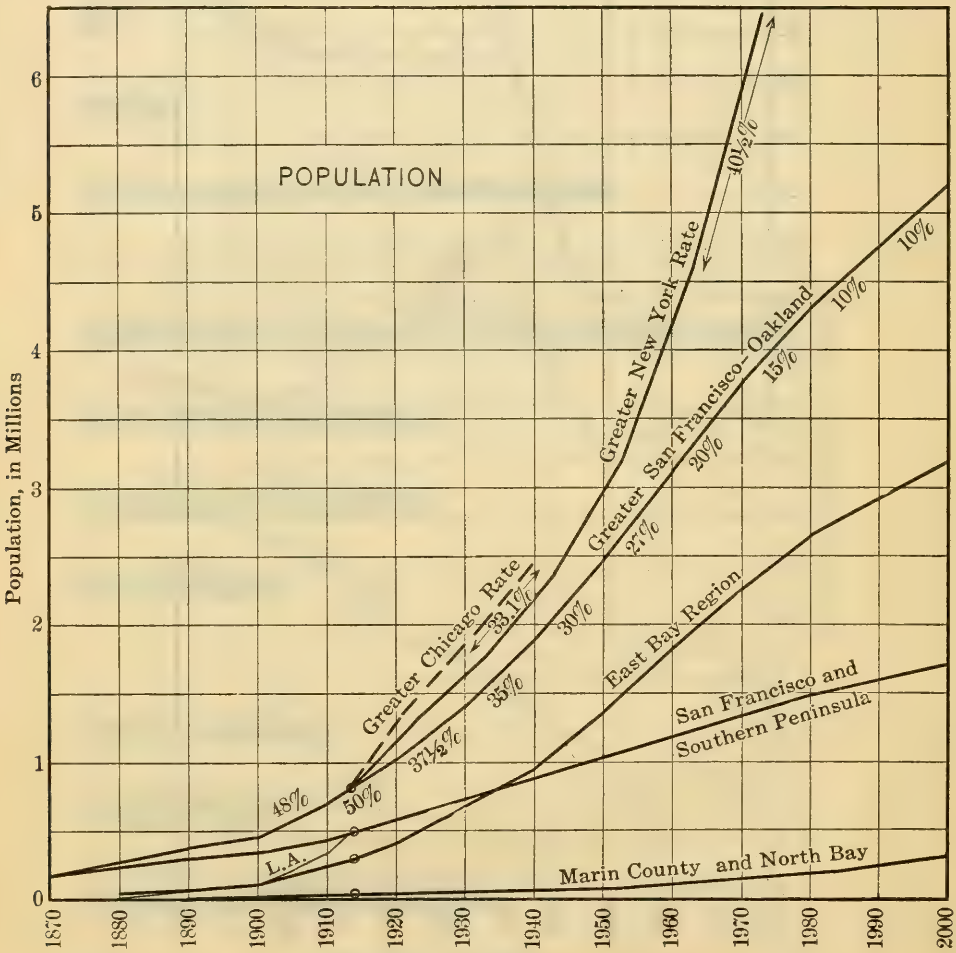


FIG. 10.

strength, and Mr. Epes Randolph, now of Tucson, Ariz., outlined the broad general plans. It is the most complete, extensive, well-constructed, and well-equipped system of interurban electric railroads in the world. There are 6 200 daily train movements over 47 routes, the total number of passengers carried yearly being more than 106 000 000. The total annual trans-bay suburban traffic between San

Francisco and Oakland, Berkeley, etc., is less than 35 000 000, or about one-third, in spite of the fact that such suburban service is, next to that of Los Angeles, the best and cheapest on the Continent.

In addition, there is doubtless no city in the United States—and that means the world—with a system of surface electric roads as modern and well-equipped as Los Angeles. The street railway situation in San Francisco is deplorable.

3.—The *Los Angeles Times* and its owner, Gen. Harrison Gray Otis, have had a tremendous influence in all Southern California, particularly Los Angeles, in maintaining the "open shop" and keeping down the cost of manufacturing and building. High wages, generally, are more than desirable, but when shop costs are greater in one center than in others near-by, the effect on local development is very bad.

4.—The ocean beaches of Southern California constitute a tremendous attraction for the class of people who come to California. It is true that there are no natural harbors in the South comparable with the marvelous one of San Francisco Bay, but the San Diego Harbor will soon admit vessels of 35-ft. draft, and Los Angeles is building an artificial harbor, jocularly referred to as a "harborette," which, nevertheless, will afford shipping facilities sufficient to cause competing points to realize thoroughly that Los Angeles is a seaport. Furthermore, the San Francisco port charges and regulations are onerous, and the cost of repairs to shipping so high that, not infrequently, vessels go with water ballast to Puget Sound points for work to be done.

5.—An essential feature of Southern California is that it is "cheap." Los Angeles and San Diego, particularly, are frequently said to be "cheap" towns made up of "cheap" people. Wages are comparatively low, due in part to the labor situation, and in part to the large number of people who come to Southern California determined to remain. All the larger places are surrounded by country topographically suited to indefinite urban expansion in all directions. Consequently, labor and rents are cheap. The expression "Southern Cafeteria," used derisively, has played a very important part.

6.—The fact that Southern Californians are often sneeringly referred to as "goo-goos" (goody-goody) has been an effective adver-

tisement. San Francisco, on the other hand, has been widely advertised as the "Paris of America." There are 200 saloons in Los Angeles and 2 200 in San Francisco proper, and Los Angeles has more than twice as many high school students.

7.—Until a decade ago, San Francisco was, politically, the State of California, and was practically governed by railway interests. Of all boss-ridden commonwealths in the United States, California took first rank, and bribery, "influence," and manipulation of public affairs for the Interests controlling the machine became accepted as a matter of course. The moral fiber of the community was seriously affected. The influx into San Francisco was relatively small, but the tremendous increase in the population of Los Angeles came from all over the United States, bringing a strong strain of Puritanism and higher civic standards. The result for Los Angeles is a community which can and does co-operate and trust its leaders, and secures vastly more advanced civic improvements, of which the water supply is the most striking.

The years of wrangling about San Francisco's water supply and private ownership of the present water system are not accidents, they are only some of the costs of its individualistic temperament.

8.—Because the newcomers quickly outnumbered the natives, Southern California is really not "Western" at all, but a sort of general average of the entire United States and Canada—particularly east of the Rocky Mountains—in education, political and religious ideals, institutions, and spirit. The large cost of railway transportation from New York to the Coast has kept the percentage of European immigration very low. The influx into the State naturally goes to that portion having the same civilization and ideals as the newcomers. Consequently, almost everybody in America has acquaintances, friends, or relatives in Southern California—often very many of them. The result is an "endless chain" never elsewhere equalled.

9.—This factor has been intensified by the limited areas available for intensive cultivation. No large irrigation or colonization schemes were there desirable or even possible. Small, entirely distinct and independent tracts were often settled almost wholly by people from one general community. For example, the Ontario Project, promoted in 1888 and carried to a successful conclusion by

Messrs. William and George Chaffey, formerly of Ontario, Canada, was largely settled by Canadians from that Province, and hence the name. The various settlements—Riverside, Redlands, San Bernardino, Ontario, Pomona, etc.—have retained their individuality. All newcomers have been pouring into small tracts having a gross area of about 300 000 acres, so that the real population density of the inhabited portions of the region is fairly high. The communities have been well established since 1895, and with practically no pioneering work since 1900. So far as municipalities are concerned, however, the reverse is true, in that there are unlimited areas available for urban growth in every direction.

10.—Due to the progressive spirit, the high average and extremely well distributed wealth, and the fact that the country is a tourist playground, automobiling in Southern California is more general than in any part of the world. This quickly resulted in the construction throughout almost the entire region of a system of beautiful boulevards. These have gone far to eliminate what little "pioneering" work was left to be done there a few years ago.

11.—All the scenic attractions of the Southland have been made easy and comfortable of access and brought emphatically to the attention of every tourist. Although the San Francisco Bay region naturally offers much more in every way except ocean bathing, and might be made one of California's greatest scenic assets, this certainly has not yet been done. Thus, Southern California has come to be regarded as the nation's playground and residence section *de luxe,* where the tourist spends his weeks and months while his visit to San Francisco is ordinarily of but a few days *en route.* For this reason it is extremely probable that the Panama-Pacific Exposition in San Francisco in 1915 will really benefit Southern California much more than the Bay region, especially as another peculiarly unique and attractive, though less expensive, Exposition will be held simultaneously in San Diego.

12.—California is in an earthquake region in the sense that *tremblors* are not infrequent. The damage from the shocks of April 18th, 1906, in the East Bay cities was insignificant. In San Francisco proper the chief damage was the breaking of the gas and water pipes in the filled-in areas, thus interrupting the main water supply. This caused one of the disastrous fires of history.

Property worth $350 000 000 was wiped out of existence, many of the inhabitants were widely scattered, and the normal development of the city was violently interrupted. Los Angeles, and to a much less degree, other Coast cities, gained thereby. The final effect on the size of the two cities, however, has been much less than is generally supposed. San Franciscans have nearly all returned. The financial status of many individuals was greatly altered, and the city as a whole is less wealthy than it would have been. Otherwise it, and particularly Greater San Francisco-Oakland, was benefited in many important ways. The effect of the disaster on the location of newcomers to the State was very marked for a time, and doubtless is still appreciable. However, even a great disaster rapidly fades from the world's memory. Los Angeles, quite as much as San Francisco, is in the *tremblor* district, and has fully as many tremors, and by the law of averages the northern city is the safer. Nowhere in the State has a really severe earthquake occurred in historic times; disastrous shocks are like lightning, in that they may strike anywhere. These facts are now generally understood and realized.

Class of Domestic Immigrants.—The people who come to California have not the slightest intention of pioneering, but rather of obtaining the pleasantest home within their means. This is a fundamental fact. That agricultural and suburban land values are from two to five times higher in Southern California than in the Northern sections has not yet had any marked effect, although, of course, it must in time.

It is of the greatest importance to realize that, 20 years ago, when the growth of Southern California began to be really phenomenal—not so much in percentage of population increase as in actual amount and character—the region was agriculturally and horticulturally quite fully developed. There were thrifty, well-matured groves and orchards covering more than two-thirds of the area now devoted to horticulture, with practically all the present steam railway mileage in operation, and beach, park, driveway, and scenic resort development done. In short, all "pioneering" was a thing of the past.

It was then that the future began to be discounted, boosting became general, the Pacific Electric System was built, the oil fields were exploited, and extraordinary growth commenced.

Probable Future Growth Within the State and of Greater San Francisco-Oakland.—To what extent and how quickly may these influences and conditions be expected to change, and with what results?

San Francisco is a big city, and its characteristics will change but slowly. Nevertheless, recently there have been some important forces in addition to those generally in operation. From within there are such people as the Rev. Dr. Aked, Bishops Hughes and Nichols, Jack London, Charles Tenney Jackson, Miriam Michelson, and a number of others, and the great University of California, which has the largest body of undergraduate students in America and strong graduate departments. Those parts of Greater San Francisco-Oakland having less of the Argonaut characteristics are rapidly gaining, and within a couple of decades will outnumber and constantly more deeply influence the main city. From without, the majority vote which the remainder of the State brings to San Francisco has forced woman suffrage, suppression of race-track gambling, throwing off railway political domination, etc. A red-light abatement act would have become a State law a few months ago had not a referendum petition held it up until the November, 1914, election, and San Francisco has already abolished its notorious Barbary Coast. By the initiative, next fall, California will vote on State-wide prohibition.

Thus Greater San Francisco-Oakland is quite rapidly reducing the advantages which Los Angeles urges so strongly, of a "wholesome atmosphere" for family life, although probably two or three decades will be required to secure any degree of parity in this regard between the two great centers.

It will doubtless be a decade before the Chambers of Commerce and Development Boards in Greater San Francisco-Oakland and Central California will speed up to the present Los Angeles standard. It will be at least as long before the present unsatisfactorily financed beginnings of an electric interurban system develop to the Pacific Electric standard, and longer before the natural attractions of the Bay region are fully utilized. The cost of living will probably always be lower in the Southland, and the beach advantages will never be offset, although the counter attractions of the northern Sierra Nevada Mountains will be made more and more important.

With rapidly increasing Far Eastern trade, the great San Fran-

cisco Bay must become an increasingly important factor. The route to the Orient from Puget Sound is more foggy and less safe, and from San Diego and Los Angeles, longer. San Francisco is the logical commercial center—the New York—of the Pacific Coast.

The Panama Canal will have an important effect, but for quite a few years much less than many seem to think. For a number of years other Coast cities will probably benefit almost as much as, if not more than, the Bay cities, particularly with lower manufacturing, handling, and ship repairing costs, due to lower wages and rents.

Within the last 4 or 5 years enormous areas in the great Central Valley have been thoroughly developed, from a land-and-water-company point of view, for sale in small tracts to home builders. The grand total is much more than 1 000 000 acres, equal or superior to, in every regard—except environment and convenience—and held at from one-half to one-fourth the prices of, lands with similar crop adaptation in Southern California. Unfortunately, the general idea has not been to use the land, but to build irrigation works and "colonize." Conditions have changed in the two or three decades since the Southland was developed. In 1880, 70% of the people in the United States were on the soil, to-day but 51 per cent. Subduing and farming irrigated lands is by no means a poor man's business, and in general those who have enough capital to succeed will not pioneer.

Already the large companies see the "handwriting on the wall" and are arranging to prepare, plant, and cultivate the land—develop it completely, and then sell farms and orchards, rather than raw land and agricultural and horticultural problems to individuals. A noteworthy instance is the Mills Orchard Company, which is planting about 10 000 acres of orchards, mostly lemons, at about the mid-point on the west side of the Sacramento Valley.

The State is putting $18 000 000 into a network of permanent highways, and the good roads movement is in full swing in the Central Valley.

Several careful students of conditions in California believe that the State is likely to have a single-tax law within the next decade, especially as the initiative is available. In this event the effect on the lands in question would be tremendous.

Taking all these things into consideration, it would seem that,

in about a decade, the great Central Valley will reach the point where Southern California was 20 years ago, and will develop thereafter in much the same way as the Southland is doing. Also, on account of the increasing tendency to concentrate in large cities, the effect on Greater San Francisco-Oakland will be much the same as the development of the southern country has been on Los Angeles. This in spite of the great extent of back country and the consequent development of relatively large local centers such as Sacramento.

Manufacturing on the Pacific Coast must also increase, but whether faster or slower than the agricultural and horticultural interests depends on several factors. Northern and Central California, Washington, and Oregon, particularly, can and will have more and cheaper hydro-electric power than any other place in the United States, and the rates and service will be, and already are, completely controlled and regulated by utility commissions. California has enormous quantities of fuel oil, but its cost is now equivalent to coal at about $4 per ton, and there is little probability of material reduction. The climatic conditions of San Francisco conduce to the highest labor efficiency on the Continent. The local market is important and rapidly increasing. The Panama Canal will make cheaply accessible enormously greater territory. San Francisco is the nearest point in the United States to the potentially tremendous consuming countries in the Far East.

The present high wage rates and unsatisfactory local labor situation make it unwise to expect very great increase in the near future. Nevertheless, it seems probable that the wage "differential" in favor of the other Far West cities will gradually be eliminated, especially in view of the general tendency toward State and National arbitration of labor disputes, laborers' compensation laws, etc.

It is impossible, of course, to evaluate numerically these many factors, but such is always the case in projecting population curves. Regarding Greater San Francisco-Oakland, it would appear that:

1.—There is no probability of excessive rates of growth for the next 10 or 15 years.

2.—The increase for 1910-1920 will be considerably greater than that for 1900-1910, during which the great fire occurred.

3.—The increase after about 1925 will be very rapid for several decades.

Probable Growth of Various Parts of Greater San Francisco-Oakland.—Of almost equal importance for the matter at hand is the division of the total growth between the main, east-bay, and suburban centers.

The influences resulting in the various local growths have been set forth, chief of which is available area. San Francisco has a population density of more than 10 000 per gross sq. mile, and probably 15 000 per net sq. mile. In Marin County, the area available for city development within a reasonable time zone is relatively small and scattered, so that the suburban traffic facilities are, and, for some time, will probably continue to be, below the other local standards. Nevertheless, the climatic conditions are peculiarly attractive and the region so picturesque as to assure great suburban development of a high grade, with excellent transportation service. The population down the Peninsula will be much denser, but always essentially suburban to San Francisco alone. At the south end of the Bay is San José, as old as San Francisco, about 45 miles from it and from Oakland. It is now, and doubtless for several decades will be, a local center rather than a suburb.

The trans-bay cities, Oakland, Alameda, Berkeley, etc., which are physically one city, were the result of San Francisco's natural barriers, in spite of a suburban trip so onerous as to begin to alienate them when well grown. Another vital factor is the mainland transcontinental, and the most important of the local railway terminals and shops. Fig. 9 shows the trans-bay traffic. After the disaster in April, 1906, very large numbers of San Franciscans went to suburban points for temporary shelter, and gradually came back home in 1908. Since then this factor has been unimportant.

Although the East Bay cities are growing at rates somewhat like that of Southern California, and very much faster than the main city, they are also developing a greater independence and self-sufficiency. The social and political conditions existing in these communities are essentially different from those in the main city.

A bridge from San Francisco to the mainland has often been suggested, in fact a bill is now before Congress authorizing one. The trans-bay traffic indicates that such a structure, or a "tube"—which would be much more expensive—is not economically justified, and probably will not be for a long time.

The curves, Fig. 10, show the writer's conclusions from consideration of a great number of elements, chief of which, however, are those just given. All urban population estimates for a period of more than one-third of a century have a sharply limited value. Revolutionary changes, due to inventions, different ideals and tastes—such as a real "back-to-the-soil" movement would typify—cannot be anticipated.

II.—Probable Water Consumption.

Average Daily Consumption Per Capita.—The per capita consumption of water seems to be constantly increasing in all great cities of the United States, and at a somewhat slower rate in most European cities, largely due to higher standards of living. When meters are installed, large water wastes stop, causing an immediate drop in consumption, but an increase at once begins. There is no reason why liberal use should not be encouraged. This necessity and luxury of modern life costs less than any other, when compared with the real comfort it affords. At any rate, the increased consumption in American cities has greatly exceeded the expectations of 20 years ago.

San Francisco has a relatively low per capita consumption, which, in spite of the cool summers, warm winters, and foggy climate, can hardly continue. Wages are high, and people spend freely for comforts. Outside San Francisco proper there will be a marked increase in the use of water for lawns and gardens.

John R. Freeman,* M. Am. Soc. C. E., concludes that the use in Greater San Francisco 50 years hence will be "125 to 150 gallons per capita per day, although by that time substantially every service is metered." The probable curves for all component parts and for the entire Metropolitan District are given in Fig. 11. If anything, the quantity for San Francisco proper is higher in comparison than for the other parts of the district.

Probable Gross Requirements of Greater San Francisco-Oakland and Its Component Areas.—Ordinarily, combining the population and daily per capita consumption curves gives the gross water requirement, but, in semi-humid and arid territories, this is not quite the case. In this district a considerable area is devoted to agricultural purposes, not a little of which will be used in this way,

* "The Hetch Hetchy Water Supply for San Francisco, 1912;" Report by John R. Freeman, San Francisco, July 15th, 1912, p. 79.

even when the population of the district reaches 5 000 000. Until required for city uses, these lands will be cultivated more and more intensively with consequently greater reliance on artificial and controlled watering.

The underground resources made available by wells and the local surface run-off naturally tributary to the region and which can be conserved at a reasonable cost, would be considerably more than sufficient for the arable land. However, much of such water is used in the main and East Bay cities, and it is impossible in considering the water situation of the district to separate the two kinds

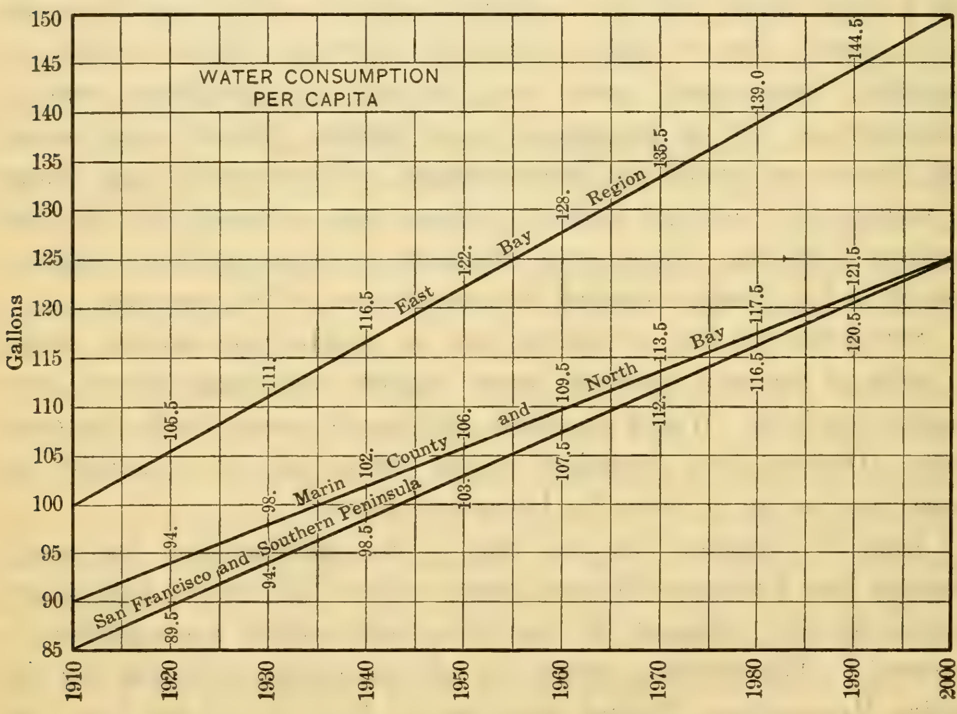


FIG. 11.

of requirements. Consequently, the easy and apparently direct way is ordinarily to obtain the gross water requirement by combining the urban and irrigation uses. Subtracting from these the quantity which may be secured locally, gives the quantity to be brought from a distance.

In doing this, however, it must be borne in mind that well water now available for irrigation and quite satisfactory in quality for domestic purposes will become unsatisfactory for the latter use without effective purification, when the region becomes urban in char-

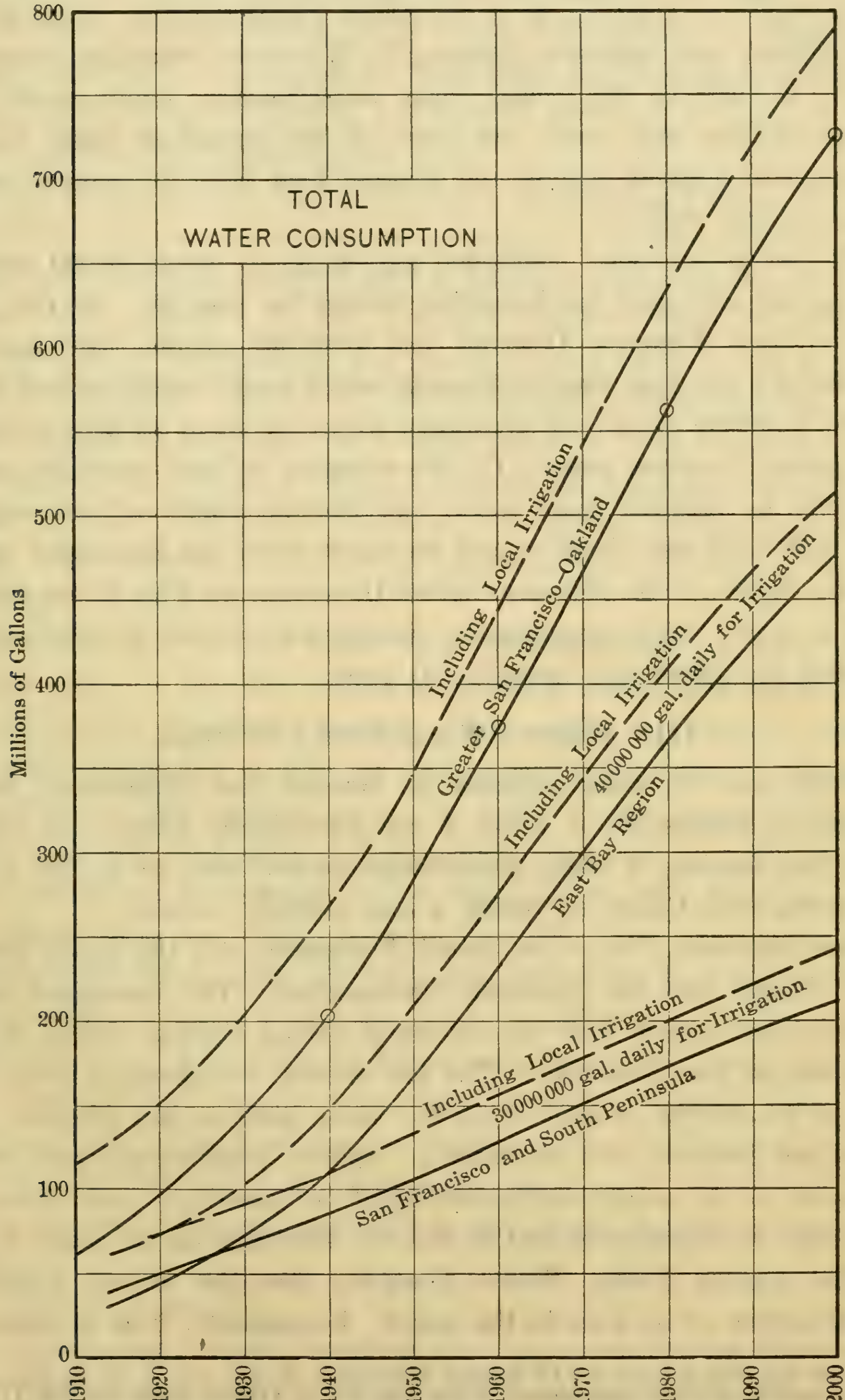

TOTAL
WATER CONSUMPTION
Millions of Gallons
800
700
600
500
400
300
200
100
0
1910
1920
1930
1940
1950
1960
1970
1980
1990
2000
Including Local Irrigation
Greater San Francisco–Oakland
Including Local Irrigation
40 000 000 gal. daily for Irrigation
East Bay Region
Including Local Irrigation
30 000 000 gal. daily for Irrigation
San Francisco and South Peninsula

FIG. 12.

acter. Hence, it is not correct to assume that the duty of water in residential city areas is, as in Southern California, the same as for horticulture and intensive farming.* Wherever irrigation water is drawn exclusively from very deep water-bearing strata, such fortunate balance may exist, but most of the irrigation water in the district and a great deal of the present East Bay city supply comes from shallow wells.

At present probably 50 000 000 gal. daily, or about 55 000 acre-ft. per annum, are used for irrigation within the district. By the time Greater San Francisco-Oakland has 4 000 000 people, this may be doubled for the area then cultivated, which could hardly exceed from 75 000 to 80 000 acres with croppings requiring about 1½ acre-ft. gross, per annum, in dryer years. Of this acreage, at least one-third would probably be watered from wells, and the remainder of the supply, say, 67 000 000 gal. daily, would be taken from the municipal water district system.† On this assumption the curves on Fig. 12 are drawn.

The grand total consumption curves are derived by adding the supplies for urban and agricultural uses.

III.—Existing Water-Supply Systems.

There are 50 water systems in Greater San Francisco; 20 in the South Peninsula, of which 8 are municipally owned; 12 in the East Bay Region, 4 being municipally owned; and 20 in the North Peninsula and Coast, of which 2 are publicly owned.

San Francisco Proper and South Peninsula.—On the South Peninsula, all but two are relatively unimportant: The municipal high-pressure fire system and the privately owned Spring Valley Water Company of San Francisco. The fire system, completed in 1913 at a cost of $5 200 000, is said to be the most modern and efficient ever built, and protects 8.25 sq. miles.‡ Special precautions were taken to make it as nearly earthquake-proof as possible. Salt water is used only in emergencies and by the two fire-boats on the water-front.

The Spring Valley Water Company has the largest privately owned system of its kind in the world. Fortunately, from a construc-

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* This was first pointed out by William Mulholland, M. Am. Soc. C. E., in connection with the estimated water requirement of the City of Los Angeles, when the Los Angeles Aqueduct was under consideration. Practically none of the territory marked out for the future development of Los Angeles secured irrigation water from shallow wells.

† C. D. Marx, M. Am. Soc. C. E., adopts 95 000 000 gal. daily for a much greater district, including a considerable part of Santa Clara County.

‡ The City and County of San Francisco have a total land area of 47 sq. miles.

tion point of view, all planning and building for a period of 43 years ending in 1911, were under one man, Hermann F. A. Schussler, M. Am. Soc. C. E. It was his rule to project his work in considerable detail for 10 years, and, true to his German technical training, he was never satisfied with "good enough work."

Distribution systems in all cities are piecemeal growths. Present conditions, such as sky-scraper construction, were not foreseen. In general, considerably different plants would be installed to-day. Due to the relief afforded by the municipal high-pressure fire system, the high class of construction, and the consistent policy of Mr. Schussler, the distribution system in San Francisco is doubtless the best, with the possible exception of Omaha, Nebr., of any important city in America, so far as it goes. Because of litigation over rates, begun in 1903, very little extension into newer districts has been made. The estimated cost of all desirable work, including extensions, up to 1920 is $2 500 000.

The company collects water from adjoining mountainous areas on both sides of the Bay, as shown in Fig. 1, and distributes 95% thereof in San Francisco proper. The quality is excellent, better than that of any city of half its size in the United States, except as to hardness. This is now about the same as the water of Washington, Chicago, Cleveland, and Louisville, and less than the Owens River supply, to be brought in by the Los Angeles Aqueduct. It will become less as the proportion obtained from surface sources increases. It will be difficult for engineers of the East and Middle West to believe that all present surface supplies come from watersheds having but 18 inhabitants exclusive of employees of the company. The company has carried water-shed protection farther than has ever been even considered east of the Rockies, purchasing outright more than 100 000 acres of water-shed land.

The present daily consumption on the South Peninsula is about as follows:

San Francisco:*		
Spring Valley Company	40 000 000	gal.
San Mateo County	16 000 000	"
Part of Santa Clara County	10 000 000	"
	66 000 000	gal.

* In addition wells and Bay supplies are used in manufacturing, to an unknown extent.

East Bay Region.—On the mainland, the important factor is the Peoples Water Company, which supplies practically all the water used in Oakland, Alameda, Berkeley, and several smaller places. Unlike the Spring Valley Water Company, it is the successor of nine lesser companies which entered the field during the past 45 years. These companies, becoming unable to meet the rapidly increasing demand, were successively displaced by stronger ones. According to J. H. Dockweiler, M. Am. Soc. C. E., who has been Consulting Engineer for the Cities of Oakland and Berkeley for a number of years, the history of these attempts to keep pace with the widely scattered and rapidly growing population centers by private capital is a most interesting and curious succession of failures properly to grasp opportunities and protect investments by adequate preparation for the future. According to him, largely as a result of 5 years' relentless and wasteful competition between two such companies, the distribution properties are in a considerable degree superimposed on each other; short-lived materials have been used extensively, and the mains and reservoirs are out of consistent relationship with the topography of the territory and the developed sources of supply. According to his records, 48% of the nearly 900 miles of pipe are 2-in. and less, and only about 30% are of cast iron. The area served is 67 sq. miles, 27 of which are above the 200-ft. contour, and it is divided into sixteen pressure zones or areas, ranging from sea level to 1 000 ft. elevation. The system has 60 000 connections, 95% of which are metered, relatively few fire hydrants, and average receipts of 25 cents per 1 000 gal., with base rates as high as 35 cents.

About 100 blocks of the business section of Oakland are covered by a municipal fire system costing, to date, $136 000. Elsewhere on the mainland the fire protection is, in general, very poor, and in many areas negligible. The situation in this regard is probably as unfortunate as in any important center in America.

About one-third the water comes from the surface supplies of the company, shown in Fig. 1, and almost all the remainder from wells near the Bay shore. According to Mr. Dockweiler, the quality is generally satisfactory, although hardly up to the standard of that supplied to San Francisco. The present daily consumption is:

Peoples Water Company:		
Surface	7 000 000	gal.
Underground	11 000 000	"
Union Water Company	700 000	"
Other systems	3 000 000	"
Private wells, about 4 000	8 000 000	"
	29 700 000	gal.

This is exclusive of irrigation water.

Marin County and North Bay Region.—The four principal systems are the Marin Water and Power Company, the North Coast Water Company, the Vallejo Municipal Water-Works, and the Benicia Water Company, the latter being privately owned. The first two are supplying about 2 000 000 gal. daily from surface sources, and the others furnish water for Vallejo (population, 15 000) and Benicia (population, 3 000).

IV.—Maximum Safe Yields of Existing Sources.

The maximum safe yields obtainable by the complete economical exploitation of the various local developed and undeveloped sources are about as follows:

San Francisco and Peninsula.—The complete development of the Spring Valley Water Company's sources of supply would require creating the Calaveras Reservoir, by building a dam 220 ft. high, work on which is now in progress; two other reservoirs, San Antonio and Arroyo Valle, all in the Alameda Creek water-shed; and the diversion of other Coast streams into the Peninsula reservoir system.

The Calaveras Reservoir, estimated cost, $1 500 000, will have a capacity of 53 000 000 000 gal., or 162 700 acre-ft. The flow-line elevation will be 790 ft., and the contributory water-shed is 100 sq. miles, or with the Upper Alameda water-shed, 140 sq. miles.

The San Antonio Reservoir, estimated cost, about $750 000, will have a capacity of 11 500 000 000 gal., or 35 300 acre-ft. The flow-line elevation will be 455 ft., and the contributory water-shed, 38.7 sq. miles.

The Arroyo Valle Reservoir, partly storage and partly regulating, is estimated to cost about $800 000, and to have a capacity of 13 000 000 000 gal., or 39 900 acre-ft. Its flow-line elevation will be 800 ft., and the

contributory water-shed, 140 sq. miles. It is estimated that the related development of underground sources in Pleasanton Valley and at Sunol will cost $1 300 000.

The Peninsula development will intercept the run-off from 65 sq. miles of territory, contributory to Pescadero and San Gregorio creeks, and convey it by canals, pumps, and tunnels into the Crystal Springs Reservoir, at an estimated cost of $8 500 000 for 50 000 000 gal. daily, or $170 000 per 1 000 000 gal. daily.

The maximum yield which could thus be economically secured for use in San Francisco proper has lately been the source of much bitter controversy. Since 1910 an extraordinary number of engineers, usually believed to be especially competent in such matters, have made examinations and have reported widely differing results, as indicated in Table 1.

There are complete records of gauge heights and draft on reservoirs as follows:

Crystal Springs	1888 to date.
San Andreas	1870 " "
Pilarcitos (Except totals only for 1889 and 1898-1902)	1867 " "
Lake Merced (With several breaks prior to 1906)	1883 " "

There are run-off and discharge records as follows:

Niles Dam	1888 to 1900.
Sunol Dam	1900 to date.
Sunol Aqueduct at Brightside Weir....	1906 " "
Pumping records at Belmont Station (With some breaks prior to 1898)..	1888 " "
Calaveras Creek at dam site..........	1898 to 1908, and 1911 to date.

Fig. 13 shows the continued run-off data at the Crystal Springs, San Andreas, and Pilarcitos Reservoirs, platted on arithmetic-probability scales, according to the method proposed by Allen Hazen, M. Am. Soc. C. E.* The consistency of the data thus shown makes striking the wide variation of engineers' estimates in Table 1.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1539.

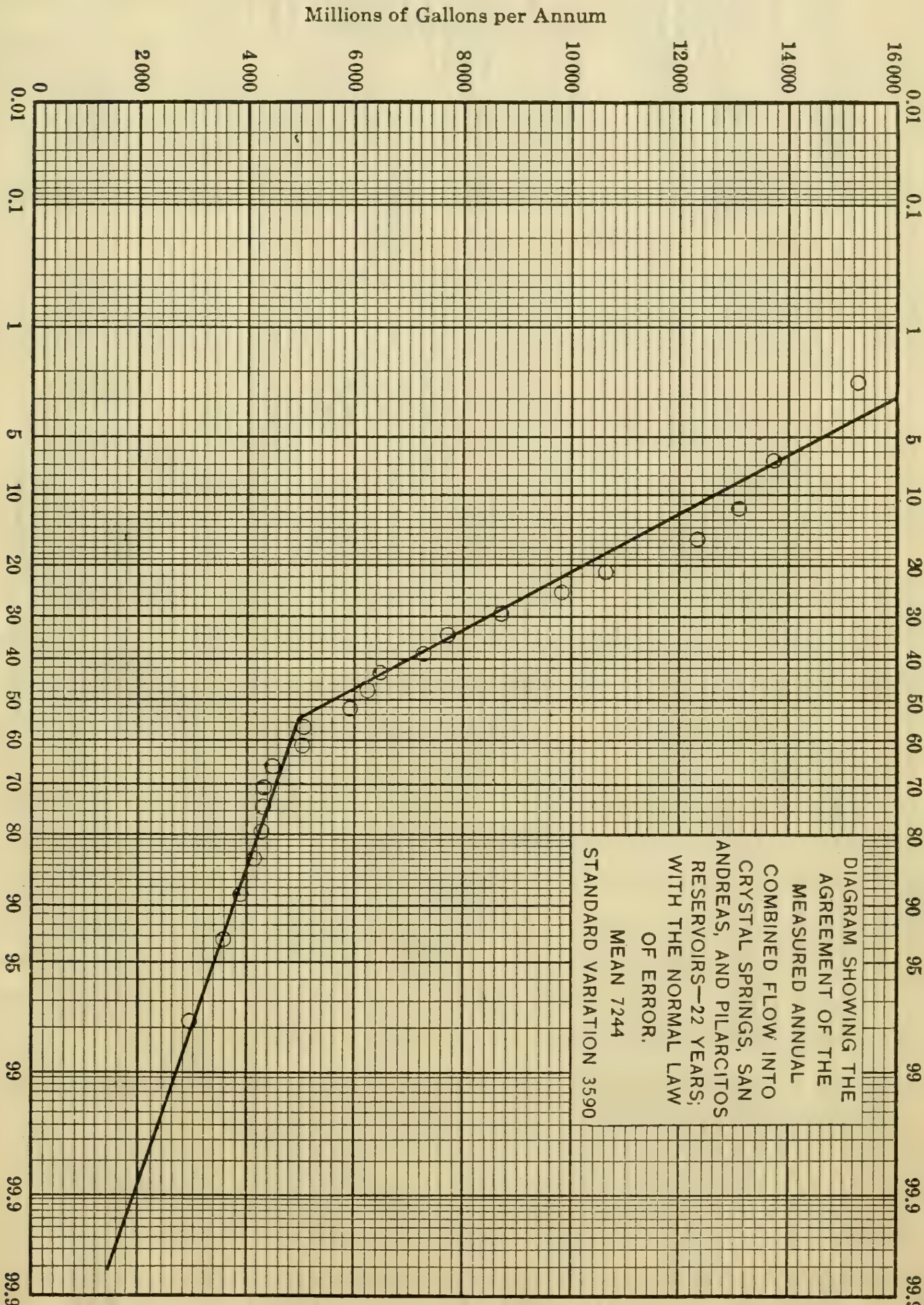
Millions of Gallons per Annum
0
2 000
4 000
6 000
8 000
10 000
12 000
14 000
16 000
0.01
0.1
1
5
10
20
30
40
50
60
70
80
90
95
99
99.9
99.99
DIAGRAM SHOWING THE AGREEMENT OF THE MEASURED ANNUAL COMBINED FLOW INTO CRYSTAL SPRINGS, SAN ANDREAS, AND PILARCITOS RESERVOIRS—22 YEARS; WITH THE NORMAL LAW OF ERROR.
MEAN 7244
STANDARD VARIATION 3590

FIG. 13.

TABLE 1.—RESOURCES OF THE WATER SUPPLY OF GREATER SAN FRANCISCO-OAKLAND.

Source.	Mendell.	Scowden, 1875.	GRUNSKY. Eng. Soc. of Pacific Coast, 1908.	GRUNSKY. Tuolumne Report, 1902.	DOCKWEILER. Long's Report to Secy. of Int., 1910.	DOCKWEILER. American Cosumnes Report, 1912.	Freeman.	Herrmann, 1912.	Anderson, 1912.	Schussler, 1912.	Mulholland and Lippincott, 1912.	Chittenden and Powell, 1912.
PENINSULA SYSTEM.												
Lobos Creek	2	2.25										
San Mateo drainage*	17			17	21†	19	20–23†	19.50	19.02	19.00		19.50
Pilarcitos		5.16										
Lower Pilarcitos		1.93										
San Andreas		4.28										
Upper Crystal Springs		15.00										
San Mateo (part of L. C. S.)		3.23										
Lake Merced		4.267		2		3		3.50	2.80	3.00		
Coast streams		58.00	20		20	31.7		51.20 (Coast streams and West Union creek)		50.00		50.00
West Union creek												
Ravenswood and Aloiso wells								21.00		21.00		20.00
San Francisquito cr. (Portola)				7.5	7	5.8						
Total Peninsula system					48	59.5		95.20		93.00		89.50
ALAMEDA SYSTEM.												
Calaveras	30‡	79.692 (Calaveras and Upper Alameda)		25–32				60.14	57.00	58.00		
Upper Alameda										10.00		
Accumulated surplus									9.48			
San Antonio				3–5				8.92	8.50	15.66 (San Antonio and Sunol drainage)		90.00 includes Calaveras.
Sunol drainage								11.36	7.40			
Sunol and Pleasanton drainage				12								
Arroyo Valle								45.38 (Arroyo Valle and Livermore Valley)	18.00	30.00	51.50 (Arroyo Valle and Livermore Valley)	40.00
Livermore Valley									30.00	17.60		
Evaporation									12.00			
Total Alameda system					57	50	Present 15.17 Future 30.40	125.80	142.38	131.26		130.00
Grand total	80§				105	109.5		221.00		224.26		219.50

* Pilarcitos, San Andreas, and Crystal Springs. † Including Lake Merced. ‡ 25 000 000 000 gal. storage. § Includes Coast streams connected with enlarged (Lower) Crystal Springs Reservoir and also surplus from Calaveras, but does not include San Antonio, Arroyo Valle, Pleasanton, or Sunol gravels.

The territory from Greater San Francisco's southern limits on both sides of the Bay to the south end of the Santa Clara Valley can be fully supplied from the remaining local surface and underground water sources within reasonable limits of cost. There will be little excess, however.

East Bay Region.—The maximum safe yield of underground sources is problematical; it depends primarily on the extent to which the various fruitful underlying water strata are supplied from local sources, such as delta fans and cones, and from distant exposed Pleistocene gravels. Considerable difference of opinion exists regarding this matter, also. Definite conclusions are not yet justifiable as to the extent to which the present draft of probably 12 000 000 or 13 000 000 gal. daily may be safely maintained or increased.

The Peoples Water Company proposes to increase the local surface water supplies by building three dams on San Pablo, Upper San Leandro, and Pinole Creeks, thus creating three collecting and storage reservoirs.

The first of these it is estimated will cost about $2 335 000, have a capacity of 6 000 000 000 gal., or 18 420 acre-ft., and a flow-line elevation of 325 ft. The water-shed is 34.65 sq. miles and the estimated safe yield 8 000 000 gal. per day.

The second reservoir, on San Leandro Creek, estimated cost $2 878 000, would have a capacity of 16 700 000 000 gal., or 51 270 acre-ft., and a flow line at an elevation of 470 ft. The water-shed is 32.94 sq. miles. Below it is the present Lake Chabot or San Leandro Reservoir from which more water is wasted than impounded in seasons of average precipitation. The upper reservoir would conserve all the run-off, and thus increase the safe daily draft about 10 000 000 gal.

The third, on Pinole Creek, the last which would be created, would cost about $676 000, have a capacity of 2 300 000 000 gal., or 7 060 acre-ft., and a flow line at an elevation of 290 ft. The water-shed is 10.5 sq. miles, and the safe draft would be about 3 000 000 gal. daily.

These proposed reservoirs are practically contiguous to built-up areas. In time their water-sheds will be built on, at least, it is desirable that the area be available for the rapid growth of population. It would be necessary to filter the water carefully. The utilization of the San Pablo and Pinole sites would affect the annual water replen-

ishment in the delta cones of these creeks where a considerable number of wells are in use. The quantity of water required to be delivered to such regions as recompense, would be about 1 500 000 gal. daily.

The Richmond Water District plans are for a distant supply from the Sacramento River.

Marin County and North Bay Region.—The plans outlined by the Marin Municipal Water District provide for a total safe draft from local sources of 14 000 000 gal. daily, which is regarded as the maximum feasible limit. In the North Bay region the available increase from local water sources is relatively insignificant.

Summary.—The total ultimate safe draft for municipal purposes, of a quality quite satisfactory and well above the present standards for important cities in America, and obtainable at a reasonable cost by complete development of local sources is, then, about as follows:

For San Francisco and Peninsula...........	180 to 200
" East Bay Region........................	37 to 40
" Marin County and North Bay Region....	14 to 16
	231 to 256

V.—Can Greater San Francisco-Oakland Afford a Sierra Supply?

San Francisco proper has available a disproportionately large part of the local water yield. By complete development, which can be done piecemeal as necessary, it can have an ample supply of excellent quality for at least 75 years to come, exceptionally large local reservoir capacity, and consequently relatively small upkeep and danger of interruption from floods, landslides, or earthquakes.

The city's present assessed valuation is $623 000 000. To acquire the Water Company's properties needed will cost between $35 000 000 and $40 000 000. The city has not finished rebuilding following a $350 000 000 fire. It has a total authorized bond issue of $43 000 000, exclusive of the Hetch Hetchy issue of $45 000 000 which latter is authorized, but not put out. Because of an exceedingly bad transportation situation, it is well started on a programme of taking over and operating all traction lines within its limits. This will require an estimated expenditure of between $100 000 000 and $125 000 000 during the next 25 years. It has no elevated or subway systems, though these are needed and cannot long be postponed. It is a rapidly growing city, with

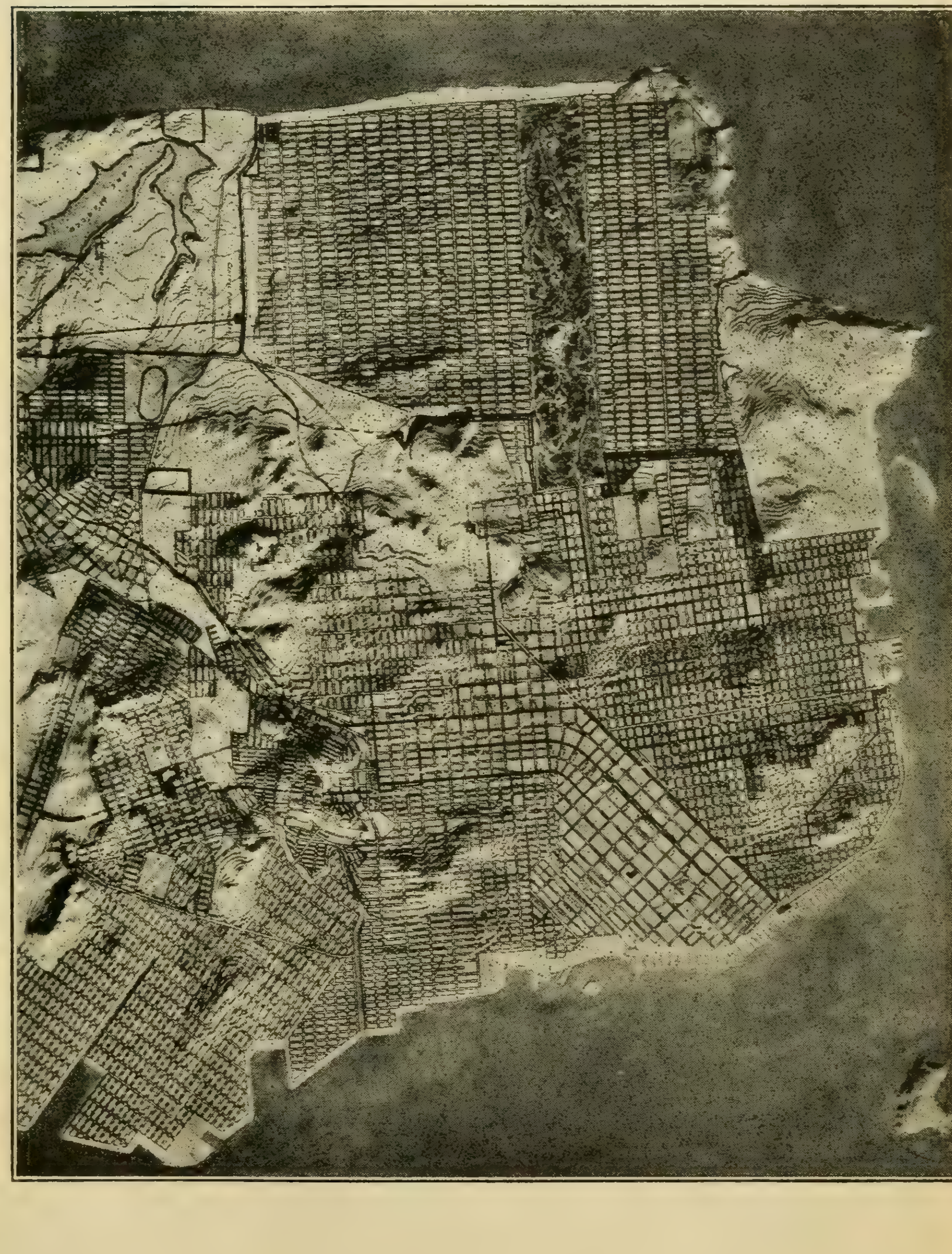

Fig. 14.—Relief Map of San Francisco, Showing Street Lines in Relation to Topography.

all the financial needs that this implies, yet is already surpassed in population by Los Angeles, which, 15 years ago, was not even admitted to be a rival. Its topography necessitates heavier costs for improvements than in almost any other city in America, and more to be done.* The local labor and political situation makes labor costs high. The temperament of its people makes concerted action difficult and progression slow and along indirect and expensive lines.

The outstanding indebtedness of Los Angeles is about 4% greater than that of San Francisco, with about seven-eighths the assessed valuation. However, Los Angeles owns its local water system, and has finished bringing in the Sierra supply. San Francisco, at least, will have to double its outstanding bond issue, in order to acquire the Spring Valley Water Company's properties, and the proposed completed Hetch Hetchy supply will add more than twice as much. At the best, then, to put the two centers on a proper basis of comparison, including the Sierra supply, San Francisco's bonded indebtedness would be more than four times that of Los Angeles. As to ability to care for such indebtedness in the future, Los Angeles is, in this regard, as in population, growing tremendously faster.

Under such circumstances, the real question is: Can San Francisco afford the luxury of a Sierra water supply at an immediate expense of more than 10% and a total ultimate cost of at least 16% of its present assessed valuation?

San Francisco's natural destiny as the commercial center of the Pacific Coast is beginning to be threatened. At least, until this matter is settled one way or another, San Francisco proper will be the largest and richest portion, and the natural leader of the Bay communities in this struggle.

In competition for newcomers, and in other elements causing a city's growth, is the water supply, as long as it is well above the American standard in quality and is delivered in ample quantities, at reasonable cost, and under satisfactory pressures, an aggressive and

* The topography and natural features of San Francisco are such that it could have been made one of the most beautiful cities in the world. Unfortunately, like many other places, it grew "Topsy fashion," and the layout is rectangular, quite regardless of the configuration of the land. There result grades as high as 22 per cent. The opportunity which the great fire of 1906 presented for re-designing the most important part of the city was not utilized. The City Engineer's office recently stated that $26 000 000 was the cost of certain improvements, necessary under the circumstances, but which would have been entirely avoided had the city's planning been adjusted to the topography, as might have been done, and that this expenditure was but the beginning of a series which from time to time must be made.

obvious factor, as compared with local transportation facilities, modern marine and railway terminals, paving, parks, boulevards, speedways, making areas spoiled by local development available for higher classes of city building, etc.?

Los Angeles, with 86% of the assessed valuation, as a matter of absolute necessity, has brought in a supply of 250 000 000 gal. per day from the Owens River at a cost of a little more than one-fourth of San Francisco's present programme. The quality of this water will, at best, be no better than the local supplies of San Francisco. Perhaps most important of all, however, is the fact that from the very outset, every drop of the water brought in will be used, and at prices well above the existing sinking fund and operation charges per million gallons daily. The reason that this is possible in Los Angeles, and utterly impossible as to Greater San Francisco-Oakland, is that irrigation is absolutely essential in the environs of the southern city, and irrigation water even at the city base rate of 9 cents per 1 000 gal., would be only $29 per acre-ft., which is well within the commercially feasible limit in that territory; on the other hand, in the environs of San Francisco-Oakland, little water is imperatively required, and none could be sold at even $5 per acre-ft. The fact that irrigation requirements for lands adjoining Los Angeles are practically the same as the urban needs will be when the city spreads out over them, has already been mentioned. If engineering is attaining the best results for the expenditures, this question is an engineering one of the highest type. It is the proper valuation, for a great city's purposes, of two sources of water supply.

East Bay Region.—The situation of the mainland cities is entirely different. The maximum possible development of the local water sources available would supply them for 10, or, at most, 15 years. Their distribution system, according to Mr. Dockweiler, is more than unsatisfactory, and their growth is already seriously hampered by the water situation. It would be advantageous for them, were San Francisco proper to "pull their chestnuts out of the fire." The rivalry between them and San Francisco is such that the latter would doubtless do nothing of the kind were the matter generally thus understood.

These communities face a serious situation. Their combined assessed valuation is only about $250 000 000. The ideal move would be for each section of Greater San Francisco-Oakland to acquire its

respective water system, extend and rebuild its distribution system, as the case may be, pool the local water supplies, and join in bringing in a common distant supply when necessary. Unfortunately, the local political and social situation makes such an arrangement improbable.

It is essential to realize that the water problem of Greater San Francisco-Oakland is very much more one for the mainland cities, than for San Francisco proper.

Marin County and North Bay Region.—What has been said of the East Bay cities is true of Marin County and the North Bay region, except on a much smaller scale.

VI.—Efforts Toward Municipal Ownership.

San Francisco, Oakland, and Denver are the only large cities in the United States which do not own and operate their water-works. According to present California laws, rates, extensions, financing, and service of all public utilities, outside of incorporated municipalities, are regulated by the State Railroad Commission. In November, 1914, the State will vote on the proposal to put all public utilities under the exclusive control of the Railroad Commission, and it is generally believed the change will be made.

Since 1880 municipal councils or supervisors have regulated, practically as to rates only, their public utilities. As elsewhere, this has been unsatisfactory, and at various times more or less serious steps have been taken to acquire or construct local municipal water systems.

San Francisco Proper.—The City had reports made by the late Gen. B. S. Alexander, Corps of Engineers, U. S. A., and the late George Davidson, Hon. M. Am. Soc. C. E., then of the U. S. Geodetic Survey, jointly (December, 1871); the late T. R. Scowden, M. Am. Soc. C. E. (May, 1874); and the late G. H. Mendell, M. Am. Soc. C. E., Col., Corps of Engineers, U. S. A. (1876-77). After the Mendell report the City officials tentatively offered $11 000 000 for the Spring Valley Water-Works' properties on the peninsula and the Company asked $12 500 000. The negotiations were dropped at this point.

On January 8th, 1900, a new charter went into effect, in which was declared the purpose of gradually acquiring the City's public utilities. Under it, the City Engineer at that time, C. E. Grunsky, M. Am. Soc. C. E., prepared plans for an entirely independent system, taking water from the Tuolumne River in the Sierra Nevada Moun-

tains, generally known as the Hetch Hetchy project. He estimated the cost of bringing 60 000 000 gal. per day to the Bay at $31 603 330 and the distribution system, at $8 807 000. After 12 years of effort on the part of the City the necessary rights of way, under severe restrictions, have recently (December, 1913) been granted by Congress in the passage of the so-called Raker Bill. On January 5th, 1914, the City officially accepted the grant and the conditions it carries.

In 1909 the voters passed on an alternative proposition for a bond issue of $45 000 000 to build the Hetch Hetchy project, including a distribution system, or an issue of $65 000 000 to bring in Hetch Hetchy water and buy the Spring Valley Water Company properties for $35 000 000. The latter lacked about 1 100 votes of the necessary two-thirds majority, and the former was carried.

On May 11th, 1908, the Secretary of the Interior at that time, the Hon. James R. Garfield, issued what is known as the "Garfield Permit," granting the rights of way requested by the City in the Yosemite National Park. This required the development of other portions of the project before utilizing the Hetch Hetchy Valley, a wonderful canyon of rugged beauty second only to the Yosemite Valley. On account of many protests, Secretary Ballinger, in March, 1909, acting on the advice of a committee consisting of Dr. George Otis Smith, Director of the U. S. Geological Survey, and L. C. Hill and E. G. Hopson, Members, Am. Soc. C. E., Supervising Engineers of the U. S. Reclamation Service, ordered the City to show cause why the Hetch Hetchy Valley should not be eliminated from the permit. A board of Army Engineers was appointed by President Taft, to assist and advise the Secretary in the matter. The members were Col. John Biddle, Lieut.-Col. Harry Taylor, and Col. Spencer Cosby, Members, Am. Soc. C. E. The appointment of this Board was probably due to President Taft's well-known penchant for the Army. Doubtless, none of these military engineers would urge his qualifications by training or experience as an expert in irrigation, hydroelectric, or municipal water supply matters.

At the request of the City Engineer, Marsden Manson, M. Am. Soc. C. E., Mr. John R. Freeman was called in by San Francisco to assist it in presenting its case. He associated with him eleven other members of the Society and President J. C. Branner of Stanford Uni-

versity. Afterward, M. M. O'Shaughnessy, M. Am. Soc. C. E., succeeded Mr. Manson (September, 1913), and shouldered much of the responsibility. Mr. Freeman fundamentally changed the Hetch Hetchy project as theretofore outlined.

The hearing was held by the Secretary of the Interior, the Hon. W. L. Fisher, on November 25th to 30th, 1912, and it is greatly to be regretted that the proceedings have not yet been published. The Army Board presented a report, largely the work of H. H. Wadsworth, M. Am. Soc. C. E., Assistant Engineer, Rivers and Harbors Branch, Corps of Engineers, U. S. A., on February 19th, 1913. On March 1st, Secretary Fisher decided that he was not authorized to act, and recommended Congressional action. His successor, Secretary F. K. Lane, concurred in this. Hence, there followed the introduction and passage of the Raker Bill.

On December 31st, 1913, the City of San Francisco filed suit to condemn the properties of the Spring Valley Water Company. A year before, tentative negotiations had been broken off, with the Company asking $37 500 000 and the City offering $37 000 000.

East Bay Cities.—On the mainland, Oakland and Richmond are the only communities which have made definite efforts to acquire water systems and supplies.

Oakland.—In 1902 Rudolph Hering, M. Am. Soc. C. E., made a report for a Citizens' Committee on the water supply for Oakland. He advised that 15 000 000 gal. per day of underground water could be obtained from the delta cones of Alameda Creek. In 1903, Desmond FitzGerald, Past-President, Am. Soc. C. E., reported to the same committee, and the next year a Board of Engineers, consisting of A. M. Hunt and J. M. Howells, Members, Am. Soc. C. E., and Mr. F. C. Turner, reported to the Oakland authorities on the adequacy of the Bay Cities Water Company's project for a local surface water supply from near Mt. Hamilton. A bond issue for taking over and building this project was defeated by the voters in December, 1904.

In the same year the rates of the Contra Costa Water Company (since absorbed by the Peoples Water Company), in Oakland, were fixed by the City authorities at a point so low that the Company appealed to the Courts. The result was the same as in San Francisco, namely, expensive litigation still pending and inadequate extensions of the system.

In 1910 litigation was begun between the Peoples Water Company and the City of Berkeley, but it was soon compromised by minor concessions on the part of the Company.

The formation of a municipal water district is now being considered.

The Richmond Municipal Water District.—This district was formed by an election held December 3d, 1912, covering the region bounded on the south by the Alameda-Contra Costa County Line and on the east and north by an irregular line approximating the eastern edge of the San Pablo Creek water-shed. It contains 56 sq. miles, of which 40 sq. miles, are below the 200-ft. contour. The population on June 30th, 1913, was estimated at 15 585, of which 13 575 were in the City of Richmond. The present assessed value in the district is $19 400 000.

The present daily water consumption is 3 400 000 gal., equal to 194 gal. daily per capita, of which 123 gal. are used in manufacturing and industrial pursuits. All water is pumped from underground sources, and almost exactly one-half is from private wells, the draft from which has about reached the available limit. The future per capita use will probably increase to 230, and then gradually drop to 150 gal. per day when the population reaches 150 000. Such large per capita consumption is expected because the industrial growth for many years will exceed the residence and business increase. There is no adequate fire protection.

The engineers of the district, Mr. P. A. Haviland, M. Dozier, Jr., M. Am. Soc. C. E., and F. H. Tibbetts, Assoc. M. Am. Soc. C. E., of San Francisco, have reported exhaustively on two water sources available. These are, a reservoir on San Pablo Creek, and by pumping from the Sacramento River at Toland's Landing, with an emergency intake at Antioch. The safe draft of 12 000 000 gal. daily can be secured from San Pablo Creek at a cost of about $3 500 000, and a like quantity brought from the Sacramento River, including a mechanical filtration plant, for from $4 000 000 to $5 600 000, depending on the conduit route and the materials used. The immediate development of a filtered river supply for 6 000 000 gal. daily, at a cost of from $2 300 000 to $4 100 000, depending on location and construction details desired, was recommended. The District's Board has adopted this recommendation and is arranging with

the towns *en route*, Martinez, Port Costa, and Pinole, to join in the project, and a bond election to provide funds will at once be held.

The present rates are from 30 to 35 cents per 1 000 gal., and it is estimated that, if the proposed plans are carried out, the cost will be from 20 to 25 cents, reducing in a few years, through large consumption, to 15 cents.

Marin Municipal Water District.—The present per capita water consumption is 85 gal. daily, a very small quantity for the class of urban development served. This is partly due to the actual inability of obtaining water as desired, but, chiefly to the high cost, which is 50 cents per 1 000 gal. and upward. Such high rates are in considerable measure due to the topography of the country. Early in 1912 the Marin Municipal Water District was formed, covering 115 sq. miles, in which the assessed valuation is $13 000 000. In November of that year A. R. Baker, Assoc. M. Am. Soc. C. E., was appointed Engineer, and M. M. O'Shaughnessy and Edwin Duryea, Jr., Members, Am. Soc. C. E., Engineering Advisers. It is understood that steps will immediately be taken to carry out Mr. Baker's recommendations. These are:

1.—Build a dam on Lagunitas Creek about 4 miles below the present reservoir of the Marin Water and Power Company (Lake Lagunitas), on the northern slope of Mt. Tamalpais, and create a storage capacity of 6 400 000 000 gal., or 19 700 acre-ft. In this way a daily supply of 5 000 000 gal. is expected to be obtained. The water will be taken around the shoulder of the mountain in tunnels and pipes, and delivered wholesale to the various localities from a system of main supply pipes. The total estimated cost of this first installation is $2 000 000.

2.—Take over the properties of the Marin Water and Power Company and the North Coast Water Company, and in time develop all their resources, together with the proposed new work, to the full capacity, estimated at 14 000 000 gal. daily, at a total cost of about $5 000 000.

3.—An alternative plan, not recommended, of pumping water from wells alongside the Russian River, about 25 miles from the north end of the district, and conveying it through Santa Rosa, Petaluma, and other communities *en route*, each of which would probably become customers. The cost for installations of 5 000 000 and 15 000 000 gal.

daily, from this source, would be very nearly equal to those respectively recommended.

The local distribution systems, like those of the East Bay cities, are without broad design, and leave very much to be desired, particularly in the matter of fire protection and permanency of materials.

VII.—Essential Factors Affecting a Choice of Distant Water-Supply Sources.

Far-sighted planning for a large additional supply should consider and provide for the water needs, not of the East Bay region, nor of San Francisco proper, but of Greater San Francisco-Oakland as a whole. Because there is a strong sentiment in the East Bay cities against joining with San Francisco in any political or financial enterprise, at least as long as San Francisco has a predominating influence, and because of a rapidly growing realization that in many ways the two sides of the Bay are competitive, it is possible the East Bay cities will act independently.

Quantities Required for Various Dates.—The additional quantities needed by the several portions of and for all Greater San Francisco-Oakland are about as given in Table 2.

TABLE 2.

	1920	1940	1960	1980	2000
San Francisco Proper and Southern Peninsula	10	50	100	155	200
East Bay Region	12	85	215	350	480
Marin County and North Bay Region	2	5	12	23	37
Totals	24	140	327	528	717

It will probably be economically desirable to obtain from outside sources about the quantities given in Table 3.

TABLE 3.

	1920	1940	1960	1980	2000
San Francisco Proper and Southern Peninsula	0	0	0	5	50
East Bay Region	3	46	178	312	443
Marin County and North Bay Region	0	0	1	9	23
Totals	3	46	179	326	516

Doubtless it would be profitable, certainly desirable, and at least a very nice thing, to furnish water to the urban communities along or close to the conduit or aqueduct line, and to the mutual interests of all concerned that excess water be used for irrigation along that line, until needed in Greater San Francisco-Oakland. Under existing laws, this would endanger a servitude being placed on such excess water. It must be remembered, however, that even the State Constitution can very easily be changed—especially with the initiative, referendum, and recall in effect.

Discounted Present Value Only Fair Method of Comparison.—Distant sources have the serious disadvantage that, as single-conduit installations cost much less than double or triple lines of equal capacity, larger and more expensive works than present needs demand are economically required; and interest charges on initial investments mount up startlingly. Those sources which are adaptable to gradual development have a marked advantage. Greater San Francisco-Oakland may obtain water, satisfactory as to quantity and quality, from so many different sources, that, to make fair comparisons, it is necessary to outline the expenditures of all kinds, construction, operation, maintenance, depreciation and renewals and incidental incomes, at specific dates, and reduce them to present values, using probable interest rates.

Location of Local Storage.—Another disadvantage of distant water sources is increased likelihood of damage to the conveying conduit or aqueduct. As the distance increases, other things being equal, not only is the risk of sudden interruption greater, but also the time required to make repairs, the cost of patroling, and of up-keep. To avert serious consequences in case of interruption, it is necessary to provide large, near-by storage facilities. The greater the risk the more storage is required.

Fortunately, San Francisco proper has Crystal Springs Reservoir on the Peninsula (capacity 19 300 000 000 gal., or 59 200 acre-ft.), and in a few years will have the Calaveras Reservoir (capacity 53 000 000 000 gal., or 162 000 acre-ft.), on the east side of the Bay; the East Bay cities have Lake Chabot (capacity, 4 840 000 000 gal., or 14 800 acre-ft.), with the possibility of creating the larger Upper San Leandro Reservoir (capacity, 16 700 000 000 gal., or 51 270 acre-ft.), at a cost of $3 000 000. Marin District now has Lake Lagunitas and Phœnix Lake (capacity, 382 000 000 gal., or 1 186 acre-ft.), and probably within a few years

another larger storage basin which the Marin Municipal Water District plans to build (ultimate capacity, 6 400 000 000 gal., or 19 700 acre-ft.).

The location and elevation of these local storage facilities have an important bearing on the selection of a distant source.

Earthquake Zones and Faults.—The seismic disturbances which have occurred thus far in California have been *tremblors,* and not very serious matters, provided—but only provided—their likelihood is frankly recognized, the facts concerning their occurrence thoroughly ascertained, and methods to circumvent them carefully and systematically worked out. President J. C. Branner, the eminent geologist of Stanford University, urges scientific, sane, and calm study of the "active faults" in the earth's crust in California. When these are located it will be possible to build so that when future earth slips occur, the damage will be negligible. If the wriggling line of the "1906 Fault" had been accurately known, the Spring Valley Water Company would not have had its pipe line laid over it, the water supply would not have been cut off, and the disastrous fire would not have happened.

Crossings of Waterways, Bay, and Mt. Diablo Range.—There are serious natural obstacles to bringing water from distant sources into San Francisco. With the Sierra Nevadas to the east there is the wide San Joaquin Valley, the River, and the Mt. Diablo Range. The first of these prohibits a gravity line, except by using very long inverted siphons under high pressure, the last necessitating pumping against high head over the lowest pass, or long tunneling in very bad ground crossed by "active faults." To bring water from the northern Sierra Nevadas, requires crossing the narrower but equally low Sacramento Valley, the River, and the Carquinez Straits. All Sierra Nevada sources necessitate taking the water for San Francisco proper across San Francisco Bay. With southern sources Marin County can only be supplied through a tunnel more than a mile long under the Golden Gate. The water here is 300 ft. deep, and the crossing would probably be somewhat similar to the one under the Hudson River at Storm King on the Catskill Aqueduct.

Many local engineers fear the difficulties of tunneling the Mt. Diablo Range, regarding it as a formation exceedingly difficult and expensive to pierce. Mr. Freeman, on the other hand, regards the difficulties it presents as much less than those of the Carquinez Straits

crossing, which latter he regards as offering such tremendous obstacles alone as to dismiss immediately from serious consideration all sources which would involve it.

Conservation.—There can be no doubt that supplying municipal needs is the highest use to which water can possibly be applied. That is by no means saying that other useful requirements should not be considered at all until such highest use is fully satisfied. All needs of a region should be considered in relation to all supplies, and adjustment made to secure the maximum total economic benefits to society. This is so obvious that taking the trouble to state it seems waste of time. Nevertheless, it is usually adroitly ignored and avoided for the reason that the greatest total benefits to society may mean individual lesser benefit or greater cost to a powerful interest. The total benefits and costs are rarely divided in like proportions. To this fact is due conservation and the conservation policy—now generally accepted in theory and almost as generally sought to be avoided in individual cases.

In California, as in all the Far West, because of climatic conditions, water is more important than fertile soil and strategic position. The water resources of the Central Valley and the northwestern part of California, sooner or later, will be so conserved and apportioned as to produce the maximum possible benefits. No use can be so high as to interfere permanently with, or prevent, such a result. Unfortunately, no definite steps have yet been taken toward such end, and this fact alone seems to justify the National conservation movement. It is extremely unsafe, however, for any interest to assume that such indifference will continue. The world moves and, in the Far West, it moves rapidly. Indeed, it is startling to look back only 10 years and see how the viewpoint of the general public has changed with regard to the exploitation of natural resources and how it has already been crystallized into fundamental changes in legislation. Any person, corporation, or municipality is most short-sighted to make investments without weighing carefully the probable changes in public opinion and consequent legislation, designed to produce a higher degree of intrinsic justice—at least for the period necessary to "unload" successfully.

Furthermore, municipalities are but mutual corporations. In Wisconsin, municipally owned public utilities have for years been regu-

lated as to service, rates, and accounting, exactly like privately owned and operated utilities; and Wisconsin's lead in such matters has been closely followed in the Progressive West.

Irrigation.—Next to municipal use comes irrigation. In discussing the growth within California, the agricultural history of the Central Valley has been touched on and the radical changes at present under way noted. The American, and particularly the western, characteristic attitude toward experience elsewhere, resulted in irrigationists having to learn by bitter experience the wisdom of the United States Reclamation Service's standard of satisfactory irrigation water supply. This standard is that all water users have ample water at all times when it can be used profitably. It involves storing flood-waters for use when streams are very low or entirely dry during the latter part of the summer. The California Central Valley has been almost the last part of the United States to find itself in this regard.

The tremendous value of the climate and fertile soil in the great Central Valley is still not appreciated. The crop adaptation is marvelous. It is conservative to say that 20 years hence an acre of properly irrigated bare land anywhere in the Central Valley will be worth at least $500, and citrus land in the foot-hills from $600 to $800. It is equally conservative to say that in the central and southern parts of the San Joaquin Valley, particularly, unirrigated agricultural land will be worth not more than from $20 to a possible maximum of $50 per acre.

Power.—Next in order of use is the development of hydro-electric power. California stands near the head of all States in the quantity of developed water-power, and yet only a fraction of its possibilities has been utilized. The cheapest fuel is oil from local fields, now costing about 75 cents per bbl., wholesale, equivalent to coal at $3 per ton, and the supply is by no means limitless. The tremendous economic importance of the power resources of the State is obvious.

The difference between the ultimate value to society of a natural resource and the commercial value of a development of that resource must be understood. Local and general economic conditions have resulted in smaller hydro-electric concerns gladly disposing of their output in blocks at high load factors to the large companies for about ½ cent per kw-hr. One of the largest power corporations in the State, with hydro-electric plants scattered for miles along the western slope

of the Sierra Nevadas, transmission lines gridironing the central part of the State, and a 60% load factor, is generating more than 40% of its energy in steam plants.

Indeed, there is a general misconception of the commercial value of "water rights." Every decrease in cost and increase in efficiency of steam plants reduces the relative advantages of water power. The steam turbine is a notable example. The "value of water rights" has lowered tremendously in the past few years, because of rate and service regulating commissions. The tendency is to regard a water right, not as an asset, but as an exclusive license for self-constituted trustees to develop a public resource and operate it for the benefit of the public, on a percentage. This change of sentiment has practically wiped out the commercial value of a proposed project controlled by a "water filing." Companies doing a tremendous business and having ample credit can wisely carry out enormous developments and secure thereby very low costs per unit of installed capacity. The rapid development and constantly lowering cost of electric generating and transmitting equipment and the progress of the art in general are other important factors.

For example, one of the recent large projects in the State, now more than half completed, is a development in which water will be dropped through six power-houses and generate 165 000 h.p., with a total cost of $130 per installed kw. or $97 per h.p. Even this will be reduced, because water from the lowest plant will irrigate a large body of land and the State Utility Commission will certainly take irrigation water rentals into account in fixing power rates. Obviously, such developments affect the value of all hydro-electric plants and projects looking to San Francisco as a market, if not throughout the whole State.

This matter has been considered at length because the Army Board evaluated the "incidental power possibilities" of the Freeman Hetch Hetchy project—115 000 h.p.—at $51 100 000. This is more than $445 per h.p. On such a basis, the water rights of the power company's complete development, just mentioned, would have to be $37 425 000. A completed hydro-electric installation feeding into a completed transmission system supplying an actually developed market, is probably worth not more than $125 per h.p.—without such market, particularly, and the means of serving it, much less. Los

Angeles is discovering this in connection with its aqueduct power, as the city itself finds it impossible to use the hydro-electric power at a high average load factor.

Navigation.—The Sacramento River is technically navigable to Red Bluff and the San Joaquin River up to Firebaugh. Both streams are much used in the lower stretches—the former to Sacramento and the latter to Stockton. Beyond these latter points navigability is at present important chiefly on account of holding down local railway rates. Consequently, navigation in this case ranks below irrigation and power uses.

Obviously, any water storage which increases the natural flow at low-water stages improves navigation. Diversion from tributaries and upper reaches of main streams for irrigation is usually, though not always, conflicting. Diversion from higher areas, entirely outside the water-shed, as a supply for Greater. San Francisco-Oakland, would always be conflicting, though, with some sources, with very small low-water flow, negligibly so.

In the Sacramento Valley there are several available storage possibilities by which the flow of the Sacramento River at Red Bluff could be increased by 500 sec-ft. for 4 months, and numerous ones to provide such increase below Marysville. In this way, navigation interests could easily and relatively cheaply be protected to permit diverting enough water from any part of the Valley drainage to supply Greater San Francisco-Oakland.

In the San Joaquin Valley the situation is not so fortunate. There the water supply is so deficient, except possibly in the northern end, that the use of storage opportunities in this way cannot be afforded.

Flood Protection.—Last of all—so far as storage is concerned—comes flood protection. The 1 750 000 acres flooded in the Central Valley must always be protected by levee systems almost exclusively. Of course, storage for almost any use would help.

Senator Newlands, the author of the Reclamation Service Act, has recently introduced a bill in Congress providing $60 000 000 per year for 10 consecutive years for flood protection, of which one-twelfth is for the Sacramento and San Joaquin Rivers, and the California Reclamation Board has outlined a $50 000 000 project for the Sacramento Valley. Spending such tremendous sums for reclaiming

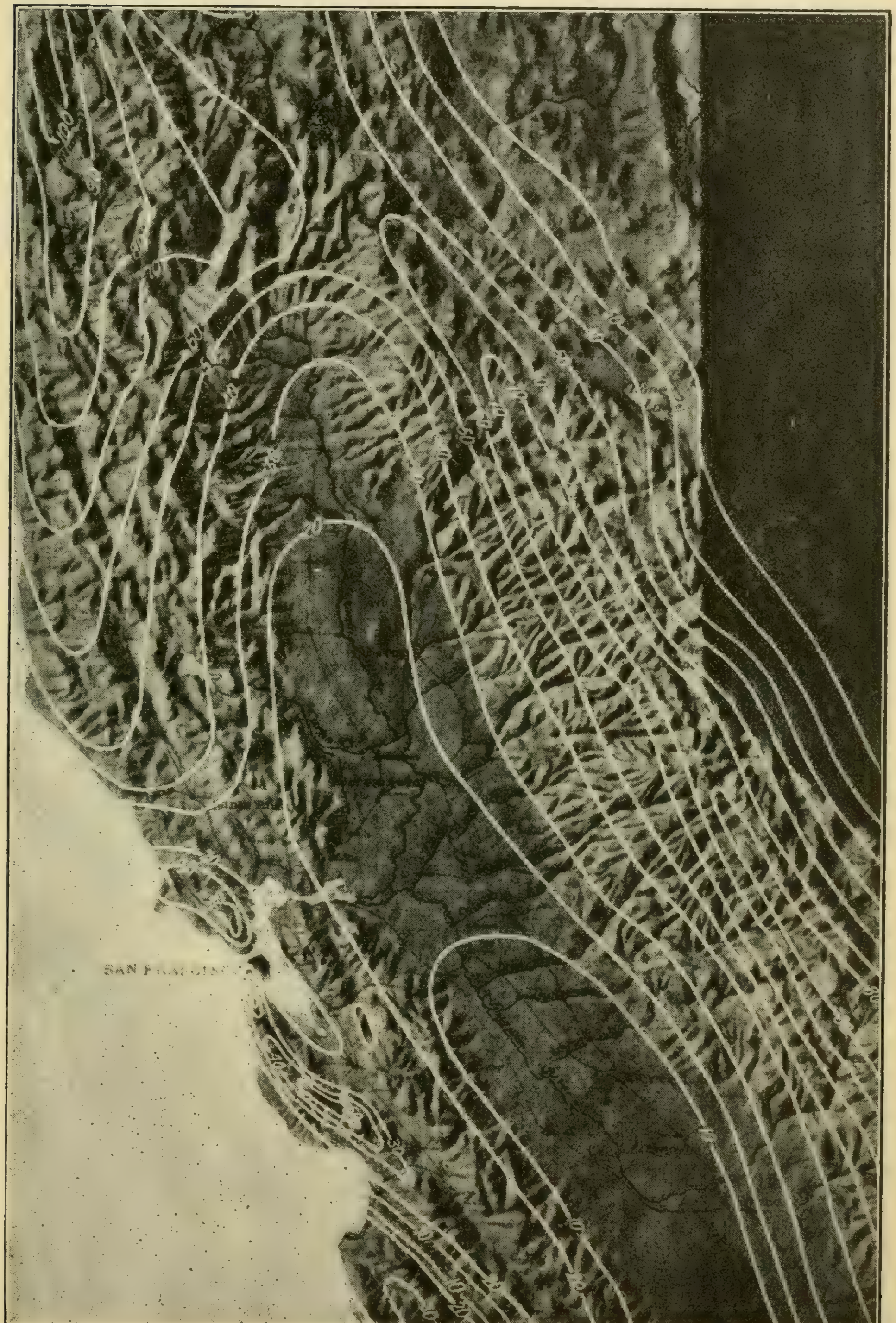

FIG. 15.—RELIEF MAP, SHOWING PRECIPITATION OVER CENTRAL CALIFORNIA.

FIG. 16.—RELIEF MAP, SHOWING RUN-OFF OVER CENTRAL CALIFORNIA.

bottom-lands and neglecting to prevent waste of such citrus lands through unwise distribution of water resources is foolish.

All Uses Interwoven.—All these utilizations of the Central Valley's water resources are inter-dependent. To secure the greatest total benefits to society offers no serious problems in statecraft or engineering —only an intelligent general plan. In some sections there will then be water to waste, in others a deficiency. In any event, all comers, no matter of how high or of how low degree, or of what precedence their claims, will sooner or later be made to conform with such a general plan.

California Precipitation and Run-Off.—Figs. 15 and 16 show the probable mean precipitation and run-off curves for the State, based on all the data yet collected.

It is more than unsafe to design hydraulic works in California from mean annual figures, because of dry cycles often 6 and 7 years long. The precipitation, speaking generally, is due to storm centers coming over the Pacific Ocean and passing on to the east. When the center enters the United States near Puget Sound, practically no precipitation results in the southern half of the State, but when the path lies farther south, say, at San Francisco, the entire State receives rain. For some reason these storms often pass well to the north for several successive years. Each year in addition, there are usually three or four relatively unimportant storm centers which pass over or near San Diego. Nevertheless, it is said that the records, kept by the Franciscan Fathers at the Mission in Santa Barbara for more than 130 years, show 1 year with absolutely no measurable precipitation.

As a consequence, in the southern end of California all significant storms are general, and in dry years so gentle and distributed that the percentage of run-off is almost incredibly small. Going northward, such conditions are less noticeable, but, throughout the entire State, especially as far north as Sacramento, long dry-year cycles frequently occur.

There are few run-off data in California prior to 1901, when the State and the U. S. Geological Survey began co-operating in systematic stream gauging. There are shorter cycles of dryer years before this period, however. This fact must be kept in mind in connection with the run-off curves given. For example, one would make a grievous

mistake by using all the stream gaugings after 1902 in Fig. 17, and assuming them to be typical. There were few reservoir records prior to 1901 in the Sierras, like those at Sweetwater and Cuyamaca.

Irrigation Water Requirements.—In determining the excess, if any, of available water above the needs for irrigation in any territory, there are the two factors, irrigable area and the local duty of water. The first is simply measured, except that in some cases the greatest utilization of a large region's water resources may require modifying the natural limits, such as water-sheds.

The local duty of water, however, is as yet a matter of judgment. It varies with soil types, present and future crops, drainage, character of supply, metered or unmetered service, and extent of cultivation. Very little Middle and Northern California data are enlightening, because very few supplies are really what they should be. Water rentals on a quantity basis are the rare exception. The almost universal disposition of irrigators to use too much water is well known. Few supplies are dependable, particularly during the latter part of the irrigating season, when streams are at the lowest. This results in using excessive quantities during times of plenty, to store water in the soil, a wasteful plan, unsatisfactory in many ways. Thus water-logging and temporarily ruining large areas is the general experience in almost all western irrigation projects. Averages of current practice, therefore, are thoroughly unsafe guides; the extreme limits are too far apart.

The various State Agricultural Experiment Stations and the U. S. Department of Agriculture have made numerous experiments to check such collected data. Unfortunately, these data do not have great practical value, just as the results of acceptance trial tests on steam-power plants are not safe bases for estimating output cost per horse-power per annum.

An important economic factor is the degree of soil cultivation. When the Sweetwater Reservoir was empty and the precipitation very small, citrus trees near San Diego were kept alive with only 3½ in. of irrigation water per annum, but at a cultivation cost normally prohibitive. Dry farming could be practiced successfully over vast areas where it is economically unjustified under conditions now and for a long time obtaining. The two greatest problems in America are to stop the disproportionate growth of cities and to improve the labor situation on the farm. Assuming that 1 acre-ft. of water per

annum and a cultivation cost of $25 per acre will produce as much as 3 acre-ft. of water and a cultivation cost of $5 per acre, is the water saving justifiable? Except where the water supply must be extended

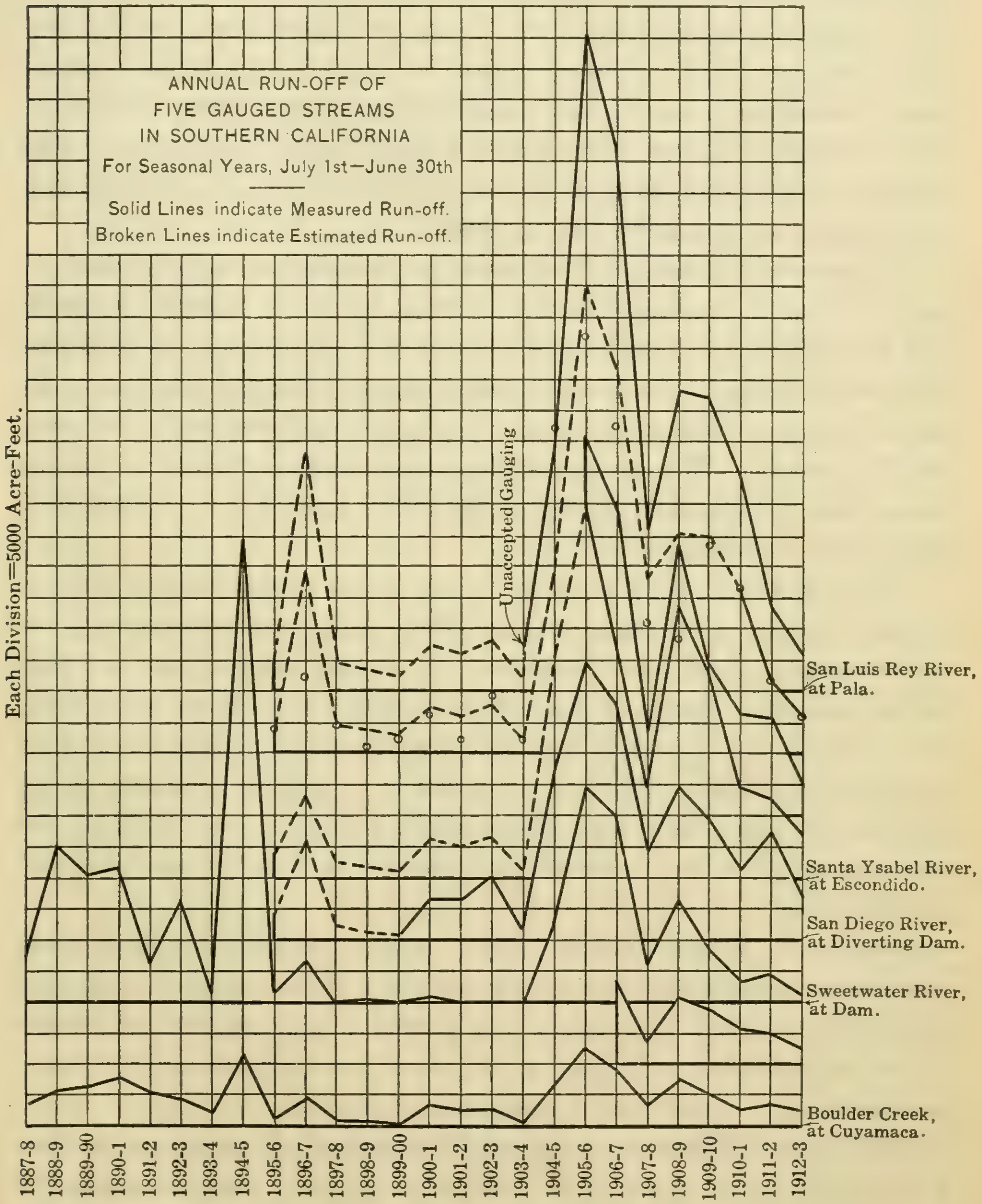

FIG. 17.

over exceptionally valuable land, obviously it is not. Experimental or theoretical and economical minimum water duties are very different things.

Many irrigation engineers believe the only safe guide is the actual water use over large successfully cultivated areas where conditions are essentially similar, where the water supply is metered, and where practically all the land is under cultivation.

It has been suggested by the engineers representing the City of San Francisco that irrigation water be used several times, that is, supply water to the land and then pump it from below the surface or collect it in drainage canals and use it again and again. The opinion of irrigation men as to the feasibility of doing this on any very large scale is particularly desired.

A great deal of land in California is irrigated from wells, particularly in the Sacramento Valley, where the water supply is ample and the underground water-table is near the surface, and in Southern California, where the surface water disappears through absorption by extensive areas of porous soils. In many localities it is the only feasible supply. The limits of the areas where sufficient irrigation water may be obtained continuously from wells have not yet been determined satisfactorily.

Value and Cost of Water.—It is important to distinguish between "value" and "cost" of many things. If there is more water in a given region than is needed for irrigation, the excess has no value for irrigation purposes—though perhaps it has for others. If there is too little, the value may be tremendous, although the cost to the users may be relatively small. When it is possible to replace available water with water which would be unsatisfactory for the use at hand but good enough for other purposes, the value element of the former would be eliminated.

Ability to Enlarge Supply Easily.—At some future time Greater San Francisco-Oakland will need more water than it is wise to develop and bring in at first. That source has a marked advantage which admits of being supplemented cheaply by other developments in the same general locality, or at least in the same direction.

Quality, and Filtration.—All available sources of supply are surface waters, and in every case the quality is, or can easily be made, far above what is generally regarded as satisfactory for large cities. Some are better in this regard, others in that; some will require immediate filtration, with others it would be unnecessary. The disposition is to insist more and more on treatment of all surface supplies, even when long

storage in reservoirs is possible. Quality standards in everything are rising, and filtration to remove coloring matter and minute organisms, even when, for hygienic reasons alone, it would be unnecessary, will probably be demanded by the public in the not distant future. Of course, the cost of filtration varies with the kind and extent of the impurities.

Sentiment.—The people of Greater San Francisco-Oakland have been taught to regard a Sierra Nevada supply as markedly better and more desirable than any other, and this fact must be given due weight.

Cost.—The cost to Greater San Francisco-Oakland is the discounted values of the expenditures required for the rights and construction of any project plus the discounted values of operation, maintenance, deterioration, and renewal charges, plus the cost of antagonistic and resentful feelings of all communities which may feel despoiled and the danger of intentional damage to much-exposed works, and minus the discounted value of incidentals such as power and water rentals.

The cost to society is the value (as the term is used in this paper) for other purposes of the water diverted. The total cost is the sum of these two.

The cost per thousand gallons delivered to him is the thing of chief interest to the average citizen, and of particular and peculiar importance to the East Bay communities. As already explained, the tide of newcomers into the State from east of Salt Lake City is almost entirely toward Southern California. If any considerable portion of this tide is diverted to the Bay regions, it will be by the East Bay communities, and, in the endeavor to do this, the striking attractiveness of all-the-year-round green lawns and blooming flowers is a vitally important factor. These media, all accentuating the California climate, have resulted in Los Angeles largely by reason of the meter water rate of 9 cents per 1 000 gal. and a flat sprinkling rate of 0.9 cent per front foot per month for all lots 150 ft. or less in depth. Less water is needed for lawns and gardens around San Francisco Bay than at Los Angeles, but, even so, the upper cost limit per 1 000 gal. at the consumer's meter cannot exceed 15 cents without having a serious effect on the area of lawn in the East Bay district.

It is a luxurious thing to have a high mountain water supply, but it is not an obvious point in competing for the Eastern and Middle West newcomers to the State. In this day and generation, to urge a

pure water supply is about as impressive as to dilate on satisfactory police protection, or paved streets—these things are assumed as a matter of course. Green lawns and blooming flowers the year round, and especially in winter, made possible by California's distinctive climate, is constantly impressive to to-day's tourists and to-morrow's fellow-citizens, and these features must be practically universal to compare at all favorably with the southland.

It is a simple matter to take a base rate of 15 cents per 1 000 gal., meter measurement, the present or prospective consumption at any given time, the cost of operation, taxes, etc., and, with any assumed rate of interest, ascertain the capital investment which may be expended on water supplies without jeopardizing this matter of lawn development. Suffice it to say that any proposed water development on a large scale, which would make water cost more than 50% greater than the rates obtaining in Los Angeles, is practically beyond the means of the East Bay cities. This point is of little importance in San Francisco, because there are notably few and small lawns, and even with free water from now on, that city never could be made comparable in that way with Los Angeles.

VIII.—Possible Distant Sources of Supply.

The following additional sources of supply have been suggested: Eel River, Putah Creek, and Clear Lake and Cache Creek, from the northern Coast Range region; Sacramento and San Joaquin Rivers, from the delta region of the Central Valley; and McCloud River, Indian Creek, Feather, Yuba, and American Rivers, Lake Tahoe, Mokelumne, Stanislaus, and Tuolumne Rivers, from the Sierra Nevadas.

A.—Coast Range Sources.

The Coast Range sources are shown on the map, Fig. 18. The water is not needed locally for irrigation. The length of conduit is much less than that for any of the Sierra Nevada sources, and, as a supply for only the East Bay cities and Marin County, is apparently well worth more careful consideration than it has received. In the construction of the project, the Carquinez Straits crossing should be made sufficiently large to serve as well for the maximum possible additional supplies, such as the McCloud, the Indian Valley, and the Feather River projects, when any or all of these may be needed.

FIG. 18.—RELIEF MAP, SHOWING POSSIBLE COAST RANGE SOURCES OF SUPPLY.

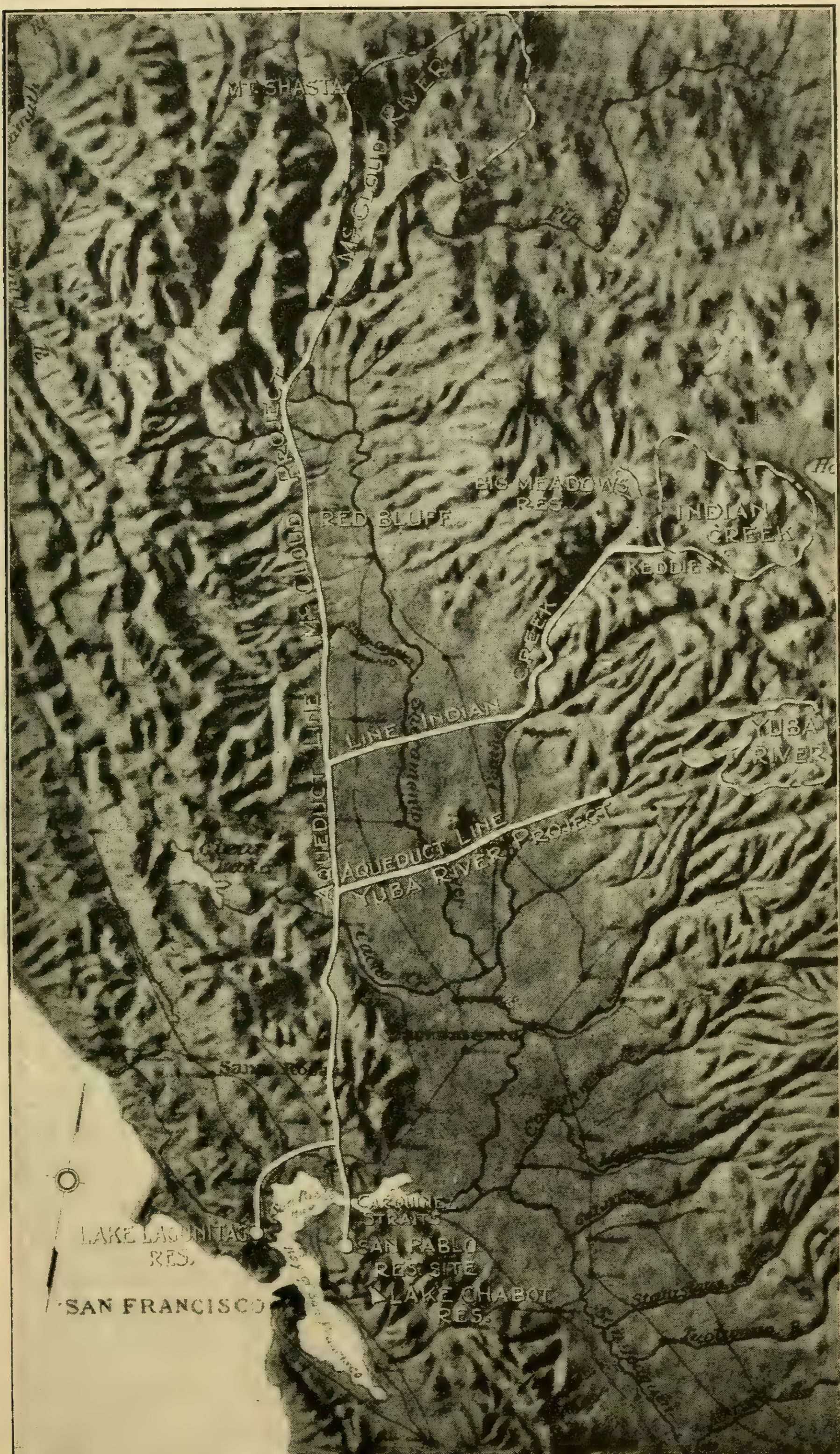

FIG. 19.—RELIEF MAP, SHOWING POSSIBLE NORTHERN SOURCES OF SUPPLY.

Eel River.—Above the Gravelly Valley Reservoir site, where 180 000 acre-ft. capacity could be created by a dam 140 ft. high, is a drainage area of 326.5 sq. miles. The safe draft is estimated at 180 000 000 gal. daily, or, with a conduit of 225 000 000 gal. daily capacity, and using the large terminal storage in Greater San Francisco-Oakland, 200 000 000 gal. daily. The conduit would be so near the hydraulic grade line that reinforced concrete pipe could be used for much of the distance. The elevation of the intake is 1 000 ft., and the length of the conduit to San Francisco, *via* Golden Gate, 125 miles. No cost estimates are available.

Putah Creek.—Putah Creek is the closest mountain source. With the excellent Monticello and Guenoc Reservoir possibilities, a safe draft of about 150 000 000 gal. daily could be secured. The conduit could follow closely the hydraulic grade line to Carquinez Straits and be 60 miles long to Lake Chabot, back of Oakland. The water should all be used locally for irrigation.

Cache Creek and Clear Lake.—Within the possible limits of lake-level regulation, 600 000 acre-ft. of storage could be obtained in Clear Lake, with which a safe draft of about 195 000 000 gal. daily would be possible. This water would not be of the best, and it should all be used for irrigating tributary valley and foot-hill land.

B.—Delta Sources.

The Delta sources are shown on the map, Fig. 20.

San Joaquin River.—The low-water flow of the San Joaquin River is so small that an intake would have to be constructed where inflow from the Sacramento River could be assured, and yet be high enough to avoid salt water from the Bay. Mr. Wadsworth suggests Clifton Court on the Old River. The supply would have to be filtered. The Sacramento River water is clearer, of better quality, softer, has less mineral matter, and a much less seasonal variation.

Sacramento River.—At the mouth of the Sacramento River the water becomes slightly brackish late in the seasons of low run-off. At Toland's Landing, 5 miles above, it is always fresh. In the future, increased irrigation use will tend to reduce the flow during the early part of the season, but will probably slightly increase the low-water flow, though storage for power development and other purposes will materially increase the low flow. It is highly improbable that sea

water will ever get materially farther up stream than at present. Mr. Allen Hazen reported in 1912 that the "Sacramento River water be filtered so as to produce a good potable water at all seasons of the year, removing the turbidity, color, and results of sewage pollution." The resulting hardness would be from 56 to 62%, or about two-thirds that of Washington, Cleveland, Chicago, Pittsburgh, Cincinnati, or Louisville, and 80% more than New York or Philadelphia.

The filtration plant and an emergency intake would be near Antioch, as planned for the Richmond Water District. The total length of the line recommended by Mr. Hazen, starting about 8 miles farther up the river, at Rio Vista, running thence directly to filtration works at Antioch, and thence to Walnut Creek, East Oakland, Alameda, and across the Bay to Potrero Point, San Francisco, is about 54 miles.

Estimates on this project were with a $3 wage scale for an 8-hour day and present railway freight charges on material from the East. Taking dates for installations of 133 333 333 gal. daily, except tunnels built originally for 400 000 000 gal. daily capacity, as 1914-18, 1936, and 1975, respectively, interest at 4½%, and assuming that all plants would begin running at capacity immediately on installation, at a cost of $17 per 1 000 000 gal. for operation, maintenance, depreciation, and renewals of plant, the present value of the cost of this source is estimated by Mr. Wadsworth as $51 700 000.

Doubtless, no one would seriously consider installing works of 133 333 333 gal. daily capacity for a community now fully supplied and using only 70 000 000 gal. daily, nor begin to operate them to full capacity immediately on completion. Caution must be used, therefore, with this figure of $51 700 000.

The water diverted would have no value for any other purpose whatever. No power could be generated, and practically no water sold *en route*.

C.—Sierra Sources.

The Sierra sources are shown on the map, Fig. 21.

McCloud River.—The McCloud River rises on the south side of Mt. Shasta, and, joining with the Pit River, forms the principal tributary of the Upper Sacramento River. It has a remarkably steady flow for California, and a minimum discharge of 1 200 sec-ft., or 770 000 000 gal. daily—amply sufficient for all possible needs. The quality is excel-

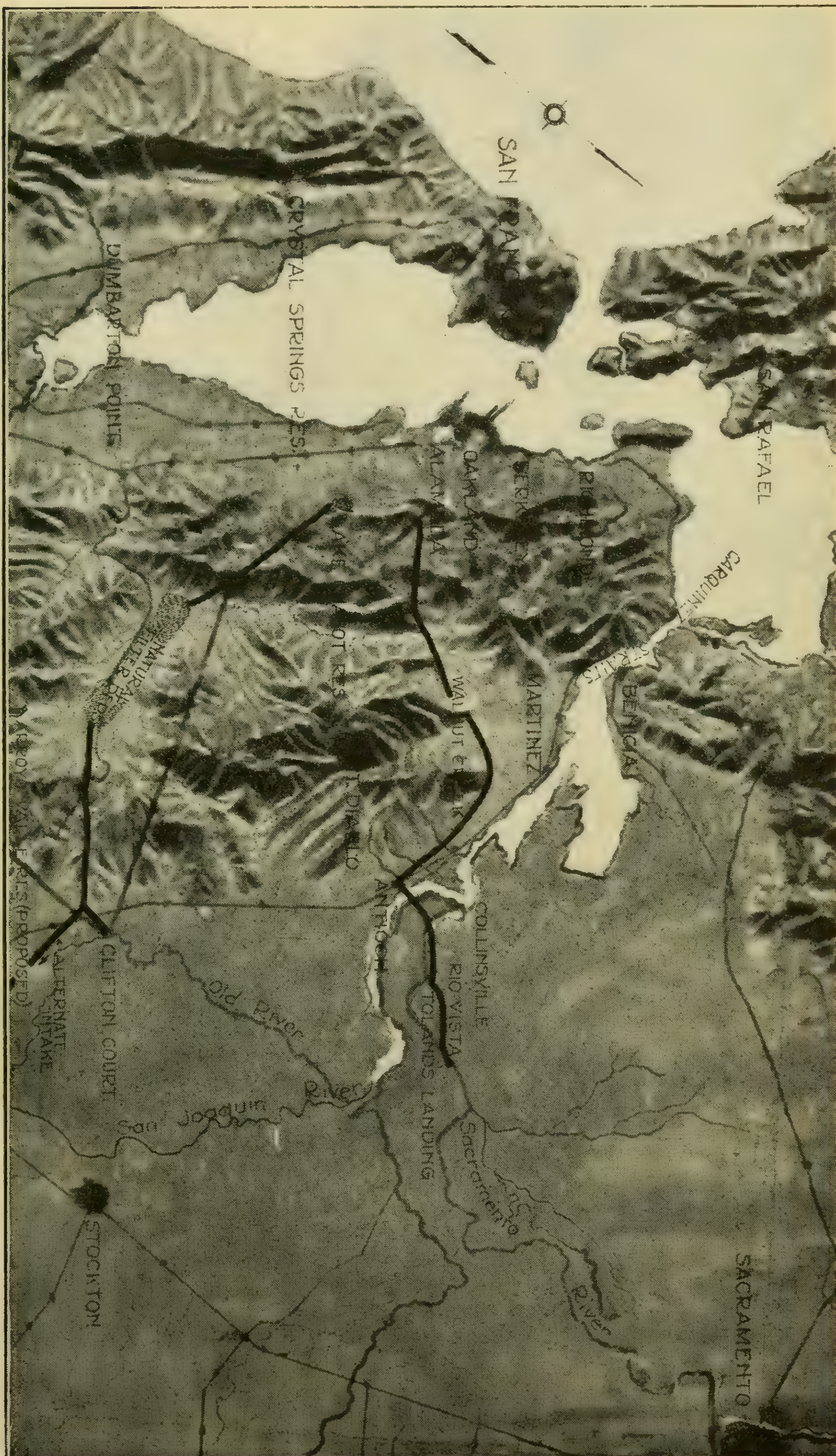

FIG. 20.—RELIEF MAP, SHOWING POSSIBLE DELTA SOURCES OF SUPPLY.

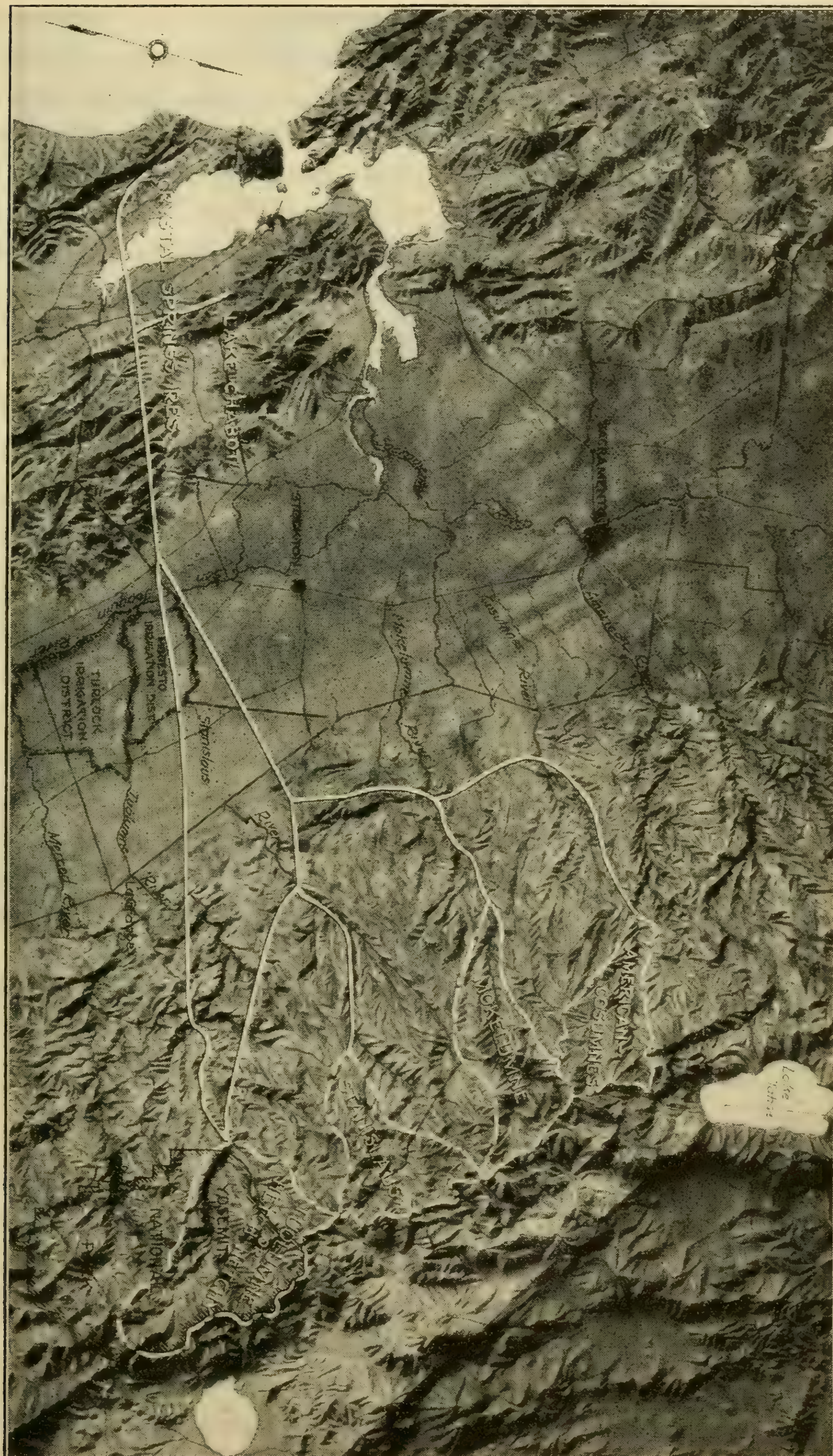

Fig. 21.—Relief Map, Showing Possible Southern Sources of Supply.

lent. Much of the water-shed is in the Mt. Shasta National Forest, and practically all the remainder, 400 000 acres, is offered for $3 000 000. All storage necessary from hygienic considerations could be had in local reservoirs. The water is not needed for irrigation, and there are storage possibilities on the river itself and elsewhere in the drainage area of the Sacramento River above Red Bluff which would more than compensate for any purpose for any quantity Greater San Francisco-Oakland could wish to divert during low-water stages. It is doubtful whether the City would ever have to do anything of the kind—certainly not for years. The comparison of this with other Sierra Nevada sources is thus practically one of cost only.

An aqueduct to Lake Chabot, in Oakland, would be 212 miles long. Mr. Wadsworth's estimate for the discounted present value of an initial installation of 260 000 000 gal. daily, 1914-20, and an additional 140 000 000 gal. daily, in 1973, is $52 500 000.

No power would be developed along the aqueduct line, but, on the project and within the controlled tributary water-shed, about 130 000 h.p. can be developed at attractive costs. The conduits, almost entirely of gravity type and of reinforced concrete, would be along the western edge of the Sacramento Valley for the entire length of the latter, and would pass close to many rapidly growing towns and irrigable land having the highest crop and fruit adaptation in the State. All excess water could doubtless be sold along the way from the very start, although not at prices fully proportionate to the cost of the aqueduct and reservoir system.

Indian Creek.—Indian Creek is a branch of the North Fork of the Feather River, with one of the best and most economical reservoir sites in America. A dam 120 ft. high would create a storage of 600 000 acre-ft. and one 220 ft. high and 900 ft. long on top, 2 300 000 acre-ft. (a capacity of 445 000 acre-ft. is equivalent to a yearly flow of 400 000 000 gal. daily). The flow line would have an elevation of 3 680 ft., and the area would be 11 500 acres. The mean annual evaporation is about 3½ ft. The run-off has been gauged from January, 1906, to date, except from January 1st, 1909, to September 10th, 1911. The average annual discharge seems to be about 480 000 acre-ft. The dam site is 4 miles, by good automobile road, from Keddie on the Western Pacific Railroad.

All the water-shed and the reservoir, covering 733 sq. miles, is

in the Plumas National Forest, and, except a total of 30 000 acres, which would cost about $1 100 000, is public land. There would be no interference with irrigation or navigation, and fewer legal entanglements than with any other Sierra Nevada or Coast source, except possibly the McCloud. A total development of 140 000 continuous horse-power is possible with the final tail-race at Elevation 1 000. It has recently been investigated by the California Débris and the Conservation Commissions for flood control. It will doubtless soon be used for hydro-electric development. It has no proponents, and no surveys or estimates as a supply for Greater San Francisco-Oakland have ever been made. The conduit line to Lake Chabot would be about 196 miles long.

By State legislation, this opportunity could be held indefinitely available for future use, at no expense except the economic waste resulting from preventing power development below 1 000 ft. elevation. The McCloud Project conduit line would be followed for about half the distance, the Yuba River conduit line would be common for most of the way.

Feather River.—The water-shed of the Feather River is characterized by steadiness (for California) and quantity of run-off, and by great storage possibilities. There is no doubt that it could furnish more water than Greater San Francisco-Oakland will ever need, in addition to all possible irrigation requirements. Unless the intake were at one of the large reservoirs in the head-waters, such as Indian Valley or Big Meadows, the purifying effect of storage would be offset by mixing with run-off from areas almost but not wholly uninhabited. If the supply is to be filtered, the lower Sacramento River, except in point of hardness, is more advantageous.

Yuba River.—There is little natural storage on the Yuba watershed, except at the highest part of the South Yuba drainage area, therefore the run-off is rapid and irregular. By a system of natural and artificial reservoirs on the South and Middle Forks, a daily supply of 165 000 000 gal. of suitable water could be obtained at reasonable cost. This could be well adapted to a combination with water from some portion of the Feather River Basin, or other sources. The water is always more or less turbid. By making a diversion near the mouth, 400 000 000 gal. daily could be secured, but at so low an elevation

that pumping would be necessary and filtration advisable. Less than one-half the quantity could be spared by irrigation interests.

All these Sierra Nevada sources would be brought into Greater San Francisco-Oakland from the north, the conduits crossing Carquinez Straits. This, according to Mr. Wadsworth, would require 5 300 ft. of tunnel and 600 ft. of shafts, and would cost $1 150 000. Piercing the Mt. Diablo Range would be avoided. With the sources following Fig. 21, there would be no crossing of Carquinez Straits, but the Mt. Diablo Range would have to be passed.

American River.—The American River enters the Sacramento River at Sacramento. The Cosumnes, a tributary of the Mokelumne River, lies just to the south, and, owing to the topography, the upper and contiguous portions of the two water-sheds have usually been considered together in outlining power and water projects. A daily supply of about 230 000 000 gal. of excellent water can thus be obtained at reasonable cost, and diverted out of the valley without important injury to dependent irrigation interests. Approximately 62 000 net continuous horse-power could be developed incidentally.

Lake Tahoe.—Lake Tahoe is only mentioned because it seems naturally to suggest itself to many. It can supply daily about 275 000 000 gal. of excellent water, but all of it and much more is needed for the Nevada desert, where, logically and properly, it should be used.

Mokelumne River.—It is estimated that from the Mokelumne River a daily supply of from 130 000 000 to 150 000 000 gal. of satisfactory water could be secured. It is said that a storage of 250 000 acre-ft. is about the limit which can be developed on the water-shed, and that there are 200 000 acres of irrigable land dependent on the stream for water. In such case it is very doubtful whether it would not be against the best interests of society to divert out of the water-shed for any purpose more than half such quantity, or possibly any at all.

Stanislaus River.—Before getting even as far south as the Stanislaus River, the excess of water over satisfactory irrigation requirements has become very small, and here probably passes through zero and becomes minus. By taking into account the water due others by appropriations, it has been computed that about 60 000 000 gal. per day could be diverted out of the water-shed. The present water laws of California are little less than absurd, and it is most shortsighted to stop with them. Taking into account ultimate irrigation

needs, instead of rights, it is improbable that any water is properly available.

Tuolumne River.—The Tuolumne River is the source selected by Mr. Grunsky, City Engineer in 1901, and, because it contemplated transforming the famous Hetch Hetchy Valley into a reservoir, is generally known as the Hetch Hetchy supply. Mr. Grunsky proposed to bring in 60 000 000 gal. per day. In 1912 Mr. John R. Freeman increased the quantity to 400 000 000 gal. per day, or 618 sec-ft., and, among other results, was crystallizing opposition of local irrigation water users and securing by them conditions protecting many of their interests.

The Raker Bill, which Congress recently (December, 1913) passed, granted San Francisco the necessary reservoir, conduit, and other rights of way in, over, and through certain public lands, the Yosemite National Park, and the Stanislaus National Forest. Following the methods of the Conservation Policy, the grant requires:

1.—Filing customary maps, observance of National Park and Forest rules and of prior valid claims, etc.

2.—No sanitary regulations over the water-shed, other than as enumerated (those necessary in any event to protect campers in the Park from having polluted water to use).

3.—Recognition by the City of the prior right of the Modesto and Turlock Irrigation Districts (acreage not to exceed 300 000) to 2 350 sec-ft. of the natural daily flow at La Grange, whenever said districts can beneficially use it, and to 4 000 sec-ft. between April 15th and June 14th.

4.—The City to sell "such amounts of stored water as may be needed for the beneficial use of the said irrigation districts" at cost, with maximum and minimum quantities thereof which can be demanded during any calendar year. These quantities are to be fixed and occasionally revised by the Secretary of the Interior.

5.—The City not to divert out of San Joaquin Valley water for other than domestic and other municipal purposes.

6.—It defines "natural daily flow" as the quantity which "on any given day would flow in the Tuolumne River or its tributaries if said grantee had no storage or diversion works on the said Tuolumne water-shed."

7.—The immediate building of Hetch Hetchy Dam, 200 ft. high, and with a base "capable of supporting said dam when built to its greatest economical and safe height."

8.—The City to sell at cost electric power to said irrigation districts and municipalities within them.

9.—The installation by the City of 10 000 h.p. within 3 years of completing "any portion of the works adapted to the generation of electrical energy;" 20 000 h.p. within 10 years; 30 000 h.p. within 15 years; 60 000 h.p. within 20 years; and all additional possible power as soon thereafter as ordered by the Secretary of the Interior.

10.—The City to construct and forever maintain scenic roads and trails (estimated first cost $1 000 000).

11.—The City, after 5 years, to pay the Government $15 000 annually for 10 years, and thereafter $30 000 annually.

12.—The City to pay all costs of investigations or decisions required under the Act to be made by the Department of the Interior.

13.—The City to convey all its lands in the Yosemite National Park and the Stanislaus National Forest to the United States.

14.—The City to sell water at cost to the United States War Department in or near San Francisco.

The City of San Francisco formally accepted this grant with all these conditions on January 5th, 1914.

Under the Raker Bill, it is impossible to estimate closely either how much water San Francisco may divert or the cost of doing so. It is to finance the complete conservation of the water from the entire water-shed above La Grange and receive thereby the excess over local irrigation needs. The City assumes all the uncertainties. These are:

1.—Quantity of water needed for beneficial use by the 300 000 acres of land during critical years. Opinions vary from 600 000 to 1 050 000 acre-ft. per annum.

2.—Quantity of stored water needed for the beneficial use of the Irrigation Districts.

3.—Quantities the Secretary of the Interior will consider proper for the annual maximum and minimum quantities of stored water the City must furnish. This will be determined by the results of experience in the matter, and by public sentiment at that time, as reflected by the Secretary of the Interior, regarding the propriety of the City

having insisted on going for its water supply to a region where there is a deficiency instead of an excess.

4.—The available storage capacity in the water-shed. This has been usually taken at 800 000 acre-ft. The U. S. Geological Survey adopts 1 000 000 acre-ft. Mr. O'Shaughnessy, the present City Engineer, estimates it as 1 800 000 acre-ft.

5.—Quantity of run-off at the various reservoir sites and diversion points during critical years.

6.—Quantity of storage which the power projects and the irrigation districts will provide without cost to the City.

7.—Construction, depreciation, and operation cost of an undetermined quantity of storage.

The amount which, if invested at 4½% compound interest, in 1914, would construct the Freeman Hetch Hetchy project is estimated by Mr. Wadsworth as $38 901 000. This contemplates the initial installation of 160 000 000 gal. per day, 1914-20 (mean date 1917); 80 000 000 gal. per day additional, 1945-47; and 160 000 000 gal. per day additional (total 400 000 000 gal. per day), 1963-69.

These figures must be used cautiously, however, because:

1.—The City has revised the estimated cost of the first 160 000 000 gal. per day installation from Mr. Freeman's figure of $37 501 400 to about $64 000 000, and probably the figures on the other installations as well.

2.—No renewals of relatively short-lived constructions have been taken into account. There are about 100 miles of steel pipe, much of it under heads as high as 800 and 900 ft.

3.—The Freeman Hetch Hetchy project and the Raker Bill Hetch Hetchy project are very different things.

Net horse-power to the amount of 115 000 can be developed from the ultimate 400 000 000 gal. per day supply, at a cost for power-house and machinery installation of $6 000 000. A large part of this power must be disposed of at cost. For the remainder, the Railroad Commission will fix the rates, at least, practically.

So much for the cost to the City. There is also a cost to society, as any water diverted out of the shed will, within 25 years at most, be needed locally for irrigation (400 000 000 gal. per day (445 000

acre-ft. per annum) is probably enough to irrigate 200 000 acres of citrus lands or 130 000 acres of valley land). Assume the cost of putting such water on the land at $125 per acre, the value of land with water at $500, and without water at $50. The value of water, therefore, would be $325 per acre, 25 years hence. Taking 175 000 acres, the total is $56 875 000, and the present value of this (interest at 4½%) is $18 924 000.

This must also be used with caution, because:

1.—It is doubtful whether the City can obtain 400 000 000 gal. per day from this source to divert.

2.—It is not certain that all the water to be diverted is really needed for local lands. The exact limits of the irrigable area east of the 300 000 acres of the Turlock and Modesto Irrigation Districts have not been definitely determined. Neither is it known how much, if any, of this can be otherwise watered, from other streams, or by pumped wells. Where this may be practicable, the cost to society is the increase in irrigation costs capitalized.

3.—There are no reliable data to determine how many acres the diverted water would irrigate, nor the cost per acre of applying it.

4.—The time in the future, when the lands would otherwise be intensively used, may not be the 25 years taken. It is certain that relatively little of the land would be properly watered immediately, because the irrigationists have not now the financial ability. It would undoubtedly be done in less than 25 years, however.

Relative to this, it is well to remember that ultimately water will be taken from the Sacramento River near its mouth and carried south to help irrigate the insufficiently watered San Joaquin Valley. When this is done, the water will doubtless be taken along the western edge of the valley, where the rainfall and run-off is only a fraction of that on the west slopes of the Sierra Nevadas. It is certain that for at least 50 miles south of the Tuolumne, as far as Kings River, no Sacramento River water will be brought across the San Joaquin Valley to supplement the Sierra Nevada streams in their respective tributary valley areas.

In any event, the present value of the cost to society of the City's selecting this supply is tremendous. This fact, more or less clearly understood, is the cause of much very bitter feeling in the Turlock and Modesto Districts. This bitterness accentuates the un-

fortunate, but in some measure justifiable, feeling common in the City's back country that San Francisco is self-centered and selfish.

Approximate Estimates.—Although there are these several elements of uncertainty, some estimates of the quantity of water which can be secured and of the cost are necessary for comparative purposes, if for nothing else. Assume that 1 500 000 acre-ft. of storage can be provided in about 40 reservoirs very well distributed over the water-shed, and that such a storage system can be operated with 80% over-all efficiency,* including evaporation during the frequent 2 to 2½-year storage periods. From the hydrograph of the Tuolumne River at La Grange, Fig. 22, it would appear that, for the 2½ years ending November 1st, 1913, the total safe draft would have been about 1 200 000 acre-ft. per annum, with all reservoirs empty at the end of the period. During the 29 months the Tuolumne River would be dry below La Grange. Another critical period would have occurred in 1897-99. Judging from the precipitation record beginning in 1868 at La Grange—which is not a satisfactory indicator of conditions, but about the only one available—similar critical periods occurred, ending, respectively, November 1st, 1878 and 1888. In other words, such cycles seem to occur about every 10 years, covering 23% of the time. The average cost of 1 500 000 acre-ft. storage capacity would probably be at least $30 per acre-ft.,† or a total of $45 000 000. Other interests might be induced to provide half of this, or incidental power might be secured which would cause an equivalent result. Consequently, the cost to San Francisco of such a quantity of storage may be taken at between $22 000 000 and $23 000 000. As the water requirements of the irrigation districts are so much larger than those of the City, and as the ultimate use will develop so much more quickly, the mean date of construction would probably be about 1930. This would make the present value of storage cost $11 620 000.

The construction cost of the initial installation, 1914-20, exclusive of the Tuolumne water-shed storage, for 160 000 000 gal. per day, would probably be about $55 000 000, according to the City's

* To secure such an efficiency it would be necessary to sacrifice most of the great power possibilities of the water-shed, because of different water requirements. For this reason, the City in 1912 asked the U. S. Forestry Service to permit no water storage in the Tuolumne drainage area above La Grange which the City would not control.

† More probably the average cost for such a quantity of storage would be more than $40 per acre-ft. capacity.

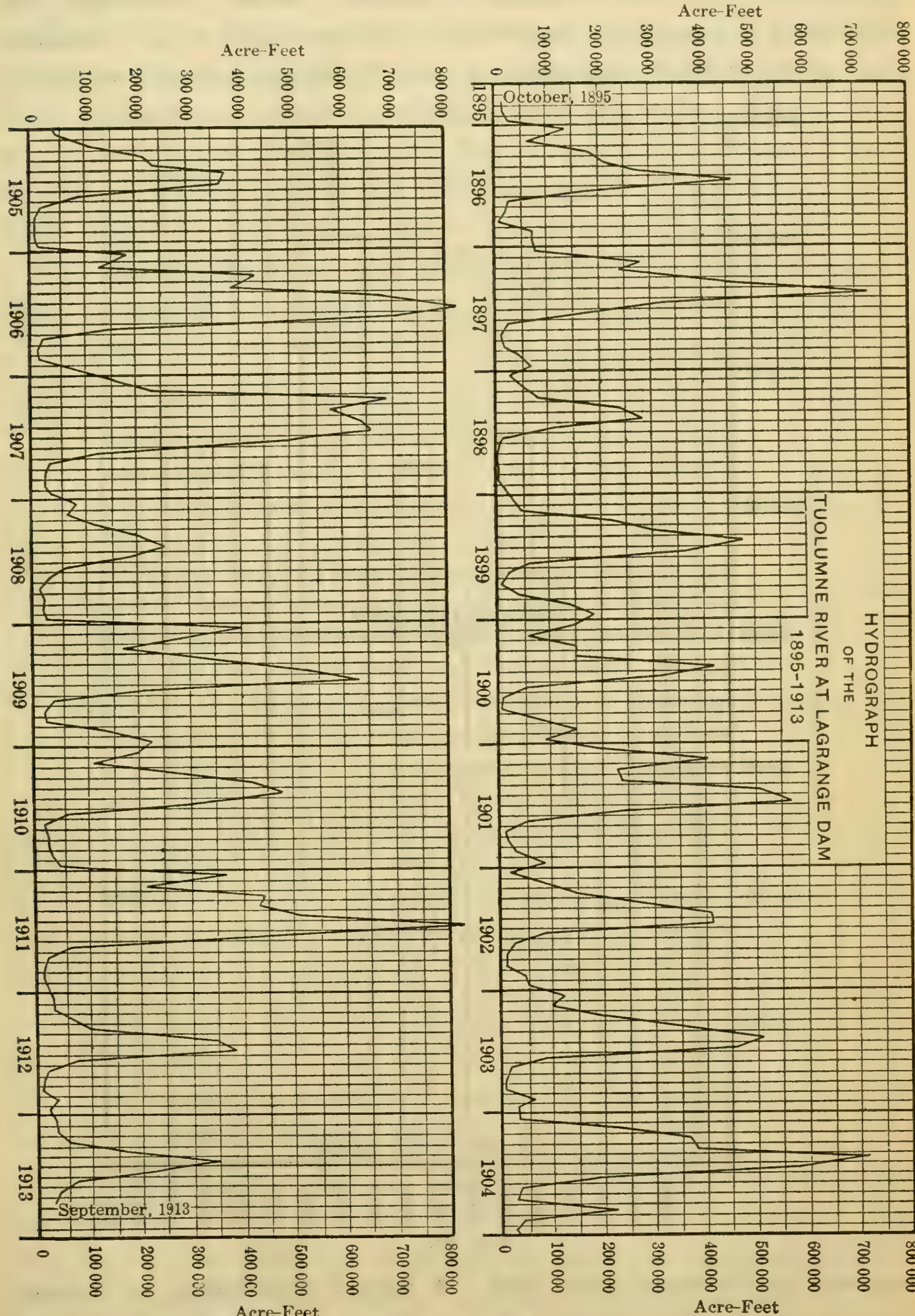
HYDROGRAPH
OF THE
TUOLUMNE RIVER AT LAGRANGE DAM
1895-1913
Acre-Feet
0
100 000
200 000
300 000
400 000
500 000
600 000
700 000
800 000
October, 1895
1895
1896
1897
1898
1899
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913
September, 1913

Fig. 22.

revised estimates. Using the consumption curves adopted by Mr. Wadsworth to secure comparative results, a second installation, increasing the capacity to 240 000 000 gal. per day, at an estimated cost of $11 000 000, exclusive of the Tuolumne storage, would be

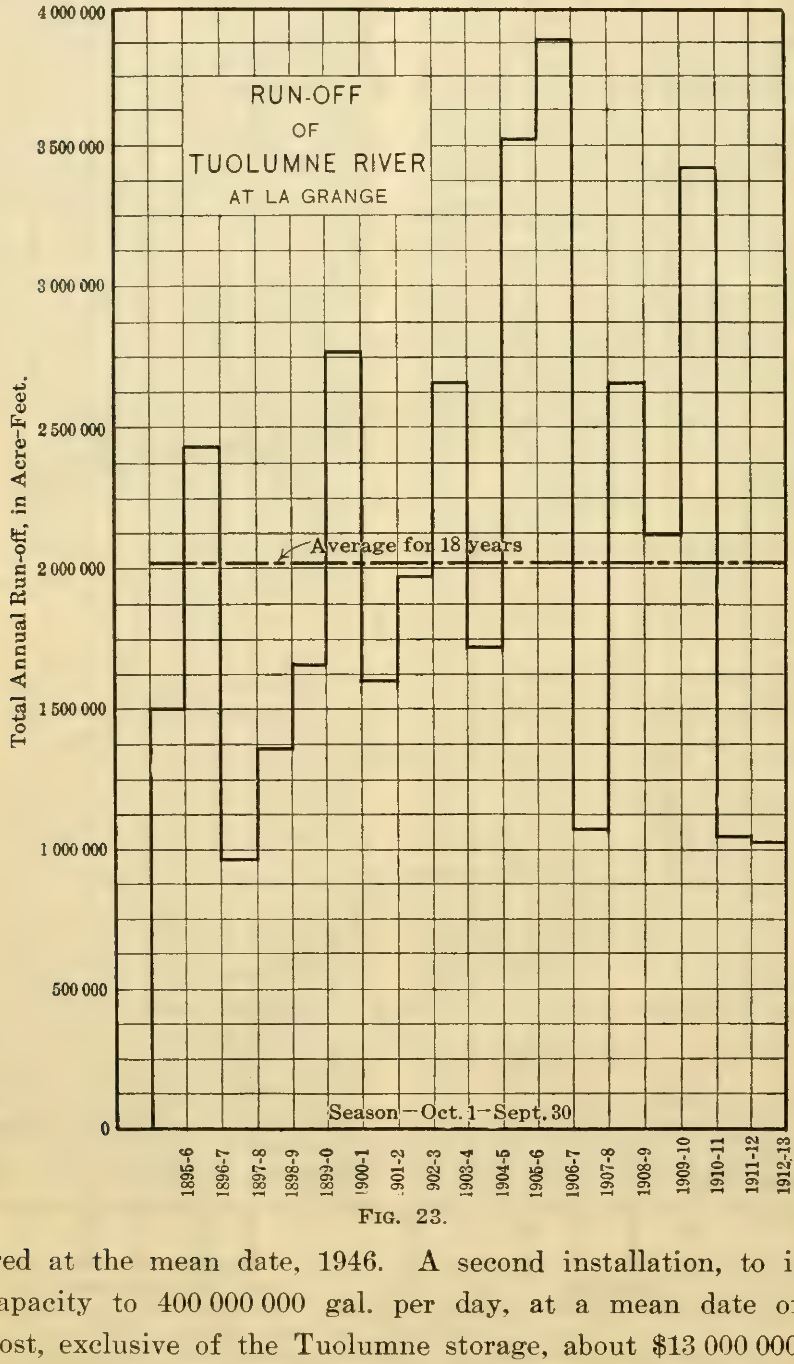

FIG. 23.

required at the mean date, 1946. A second installation, to increase the capacity to 400 000 000 gal. per day, at a mean date of 1966, will cost, exclusive of the Tuolumne storage, about $13 000 000. The present value of these expenditures is $51 650 000. Nothing is in-

cluded for renewals of short-lived materials, only initial costs. Plans which will make the Tuolumne River below La Grange dry for from 24 to 30 consecutive months, one-fifth of the time, will necessitate the acquisition of all riparian rights on the stream and many underground water rights in the region. The Wright Irrigation Law, under which the irrigation districts were formed, may have to be amended before any water can be taken out of the water-shed. The cost of necessary water rights may thus be considerable.

The value to the City of the ultimate quantity of incidental power which it has been proposed to develop can, under the conditions, be no more than $3 000 000, and the mean date about 1925. The value of power to society other than the City would probably be about an equal amount. With interest at 4½% the present value of the credit to the City is $1 848 000. Other power opportunities in the water-shed have been taken into consideration in estimating the cost to the City of storage required.

The present value of the project's cost to society, the value of the water for irrigation of lands east of the Turlock and Modesto Irrigation Districts, is, say, $17 500 000. There is a credit of, say, $3 000 000 for the proposed power installation, leaving society's net cost $15 652 000.

The grand total cost (present value) is $65 454 000.

Quantity of Water Obtainable.—The City will secure per annum whatever part of 1 200 000 acre-ft. is in excess of the irrigation needs of 300 000 acres. If such needs are 750 000 acre-ft., the City can obtain 450 000 acre-ft., or 400 000 000 gal. daily. If the irrigation needs are 900 000 acre-ft., the City can obtain 300 000 acre-ft. per annum, or 273 000 000 gal. daily, and if 1 050 000 acre-ft., then 150 000 000 acre-ft. per annum, or 136 000 000 gal. daily. The quantity available is chiefly a question of the irrigation needs of the 300 000 acres of land in question.

The writer believes that the ultimate irrigation duty of water for this land will nearly equal that in Imperial Valley, California. The two regions have much the same climate, both are essentially alfalfa and stock districts, with almost identical crop adaptations, and will doubtless have similar social conditions. The rainfall in the Turlock and Modesto Irrigation Districts averages about 9 in. and in

the Imperial Valley only 3 in., but in each case practically none occurs during the irrigation season. As an offset, the San Joaquin Valley lands are much more porous than those of Imperial Valley, which are the closest ever successfully irrigated on a large scale. All irrigation water in Imperial Valley is sold on a metered basis, and is used with care and economy. Indeed, practically no drainage system has ever yet been seriously suggested though the entire area is extraordinarily level. The irrigation canals are sealed with a deposition of fine Colorado River silt, so that the seepage loss of the distribution system is quite as low as could be expected from concrete-lined canals.* Imperial Water Co. No. 1 has 101 670 acres under actual cultivation in a compact body with the crops shown in Table 4.

TABLE 4.

Crop.	Number of acres.	Percentage.
Alfalfa	54 280	54.0
Alfalfa and barley	13 600	13.4
Barley	9 710	9.6
Corn	6 790	6.7
Cotton	6 150	6.0
Corn after barley	3 500	3.4
Melons	3 000	3.0
Miscellaneous	2 880	....
Alfalfa and corn	880	....
Vineyards	820	....

The net quantity of water actually delivered to the highest corner of each 160-acre tract of land during 1912 was 318 000 acre-ft., and in 1913, it was 325 000 acre-ft.

Safety.—As to relative safety, the presence of "active faults" in the Mt. Diablo Range, which will be crossed by the pressure tunnels, is a matter which many local engineers regard as very serious. Mr. Freeman holds the opposite opinion, and greatly prefers this construction to the Carquinez Straits crossing.

Quality of Water.—Water obtained from this source would be very soft (hardness about 20). All the contributory water-shed lies within the Yosemite National Park, and, therefore, will have no permanent population. On the other hand, with the improved transportation facilities to be made on the terms of the Raker Bill, the

* "Irrigation and River Control in the Colorado River Delta," *Transactions*, Am. Soc. C. E., Vol. LXXVI, pp. 1428-1431.

number of tourists, summer visitors, and campers on the water-shed will soon become a matter of importance. Taking this into consideration, together with the constantly rising standards in the quality of water supplies, Mr. Allen Hazen and Dr. Rupert Blue, Chief Surgeon, U. S. Navy Department, at the hearing before Secretary Fisher, in November, 1912, agreed in the opinion that complete filtration would be required in about 50 years, possibly sooner. Except for being softer, it would, under existing conditions, be almost exactly on a par with the water now delivered from Crystal Springs Reservoir. Judging from small reservoirs in the mountain regions, there is a possibility that greater trouble would be caused by minute organisms, such as algæ.

Possibility of Enlarging Supply.—If all the Bay communities pool their water supplies, distant supplies will probably not be required before 1945, and by 1990—taking the upper limit of 400 000 000 gal. per day as the quantity which may be obtained from this source—the combined sources will become inadequate within a period of 45 years from their introduction. Long before the latter date, it will be out of the question to secure any more supplies from that general region, and additional water will be obtainable only from the Sacramento River or some of its tributaries, that is, some of the northern sources, the construction of any one of which would be entirely independent of the Tuolumne works.

Combination of Sources.—Obviously, numerous combinations of the various Southern Sierra sources can be made, as well as of those of the Northern Sierras. Some of these may offer considerable advantages.

Conclusions.

Summarizing the salient features of the needlessly complexed political, sociological, and engineering problem, it appears to the writer that the citizens of San Francisco should take stock, and determine:

A.—Whether the water problem of San Francisco is primarily one for San Francisco alone or for Greater San Francisco-Oakland.

B.—Whether San Francisco should attempt to shoulder the financing and responsibility of an entire metropolitan district supply.

C.—Whether the present water system and sources supplying San Francisco proper, when owned actually by the City and properly improved and extended, are not satisfactory and adequate for the needs of the main City for the next two or three generations.

D.—Whether the other portions of Greater San Francisco-Oakland, and particularly the cities of Oakland, Berkeley, and Alameda, are not far more concerned than San Francisco in distant water supplies.

E.—Whether, on sentiment, San Francisco can afford to invest, in a new and distant water plant, sums so vast that many other requirements more vital to the interests of the City must be sacrificed or skimped.

F.—Whether the Hetch Hetchy scheme in its final form is a sound and practical one, from the engineering standpoint, and whether engineers of practical western experience, who are best qualified to pass on its peculiarly western problems, generally endorse or generally question it.

G.—Whether the Raker Act does not strip from the Hetch Hetchy project most of its advantages and assets, leaving the City in the position of "holding the bag" for others.

H.—Whether, all things considered, the only wise line of policy for San Francisco to follow is not:

First, to consummate the acquisition of the present water system, with all its appurtenances;

Thereafter, to take the water situation out of politics by having water matters handled by a Water Board of very wide powers, and entirely independent of all other City authorities;

Thereafter, to improve, extend, and amplify that system to meet fully all the City's needs for 70 or 80 years to come;

And, thereafter, to weigh carefully the relative advantages of entering into the vast Hetch Hetchy scheme or taking up some of the more important sociological problems, with a view to the more rapid and substantial development of the City itself, and let the other Bay cities work out their own water destiny for themselves.

Also, that the citizens of the East Bay cities should consider and decide, in addition to some of these same things, the following:

A.—Whether they should not acquire the existing water-works on the mainland, and at once improve, extend, and amplify them fully.

B.—Whether, as these cities are really in competition with Los Angeles rather than San Francisco, distant sources necessitating rates exceeding 15 cents per 1 000 gal. can be afforded.

Acknowledgments.

The writer is indebted to many engineers and others for valuable suggestions and other assistance in preparing this paper. It is desired to extend grateful acknowledgment and appreciation to all of them. Especial thanks are due J. H. Dockweiler and C. E. Grunsky, Members, Am. Soc. C. E., particularly for the use of private reports and data.

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PAPERS AND DISCUSSIONS

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THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA.

By E. L. Sayers and A. C. Polk, Members, Am. Soc. C. E.

To be Presented November 4th, 1914.

Synopsis.

The object of this paper is to describe the design and construction of a low-head hydro-electric plant recently completed in Alabama, and thereby bring out a discussion, if possible, on the latest ideas in the practice of developing water-power under low heads, the latest features in hydraulic turbine design, modern methods of transmission, and the construction of such plants.

The Alabama Traction, Light, and Power Company was organized in 1912 for the purpose of developing the water-powers of Alabama. The Company acquired several public service companies operating steam generating plants, street railways, gas plants, etc., and several companies owning undeveloped water-power sites. Among the latter was the Alabama Power Company, which owns a site known as "Lock 12" on the Coosa River at a central point in the State.

This plant will form the hub of an extensive system of power

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

plants capable of developing 24-hour, 80% efficiency power in excess of 500 000 h.p.

The foundations of the power-house are built transversely to the axis of the river and form an integral part of the dam. The length of spillway, power-house, and abutments, over all, is 1 530 ft. 5½ in. The power-house substructure is 303 ft. 4 in. long, 136 ft. 5 in. wide, and contains provision for six units. Four units constitute the present installation, and the superstructure is built with a temporary wall at the river end so that it can be extended to cover all six.

The spillway forming the principal part of the dam has a net crest length of 780 ft., divided into twenty-six sections, each 30 ft. long, separated by concrete piers, 6 ft. long on the axis of the dam. On the crest of each section of spillway there is placed between piers a gate, 30 ft. long and 14 ft. high, of a modified Stoney type. The height of the dam from the average elevation of the original river bed to the elevation of the flow line of the reservoir is 77 ft.

In all, there are approximately 190 000 cu. yd. of concrete in the power-house foundations and the dam. The first concrete was placed on April 12th, 1913; it was all placed by February 26th, 1914; and commercial power was turned out for the first time on April 12th, 1914, just one year, to the day, from the date of placing the first concrete.

The installation is made up of four vertical, single-runner turbines, guaranteed to deliver to the shaft, under 68 ft. head, at 100 rev. per min., 17 500 h.p. One feature of these units, as described in the paper, is that they are entirely self-contained, that is, they can be built up on an assembly floor from the foundation ring to the top of the direct-connected exciter above the generator, without requiring any lateral support.

Another feature of the development, as described in the paper, is the use of an improved volute casing, through which the water passes from the penstocks to the wheels. These scroll casings were complicated in form, and were moulded directly in the concrete of the power-house foundations, without the use of a steel lining.

After giving a brief description of the general system of the Alabama Traction, Light, and Power Company and of the power market to be served, the paper describes the Coosa River and the cli-

matic conditions under which the work was done, so that a better idea can be had by those not familiar with the locality.

The machinery is fully described, greater detail being given in the case of the turbines than for the electrical equipment, discussions of which are constantly appearing in papers of electrical societies and in the electrical technical press.

In connection with the construction of the plant, the materials used and their sources, the general scheme of handling the work and its prosecution, are dealt with in detail. The selection of plant, its layout, and the construction of camps are described, and improvements which experience showed could have been used, are noted.

The concrete forms used in the power-house foundations were necessarily complicated, and the details of their design and construction are shown. The various methods used in placing concrete and the success attending their use are described.

At the end of the paper, the design and construction of transmission lines and sub-stations are dealt with briefly.

INTRODUCTORY.

In the last 10 years the progress made in methods of generating electric power and transmitting it over long distances to markets has been very marked, and although more or less complete descriptions of hydro-electric developments have been appearing in various technical papers, there has been no extended discussion of modern methods before this Society. This fact appears the more strange because at the present time a large amount of work is being done in developing the water-powers of the United States, and this branch of the Profession is fast becoming one of the most important.

No extraordinary features are claimed for the work described. The plant develops power under the moderate head of 68 ft., the installation of 70 000 h.p. is not remarkable, and no great difficulties in construction were met. It is believed, however, that it represents the latest improvements in equipment, and that a full discussion of this development and others of similar proportions will be of practical value to the Profession by bringing before it the latest practice in developing water-power under low heads, the latest features in hydraulic turbine design, modern methods of transmission

of high-potential current, and methods of construction which have been tried and found successful.

General System of the Alabama Traction, Light, and Power Company.

The Alabama Traction, Light, and Power Company, organized in 1912, acquired the properties, rights, and franchises of several public service companies in Alabama, and of several companies owning water-power sites in the State. Among the latter were the Alabama Power Company, with several undeveloped sites on the Coosa River; the Alabama Interstate Power Company, with a site on the Tallapoosa River; the Little River Power Company, with a site on Little River; and the Muscle Shoals Hydro-Electric Power Company, owning three sites on the Tennessee River. These sites represent a possible development in excess of 500 000 h.p. continuous 24-hour power.

The developments proposed by the Company on the Coosa are at the sites selected by the United States Government engineers for dams and locks for the improvement of the river. Fig. 1 is a profile of the Coosa River covering that portion affected by the proposed developments. The dam farthest south, at which power could be developed, is at a point 7½ miles above Wetumpka. This site, known as Lock 18, has an effective head of 63 ft., and the dam would raise the water to Elevation 254 (U. S. Army datum) from Elevation 190. At Lock 15, the next site, 19½ miles above Wetumpka, the water would be raised from Elevation 254 to Elevation 294, giving an effective head of about 39 ft. At a point about 34 miles above Wetumpka, the dam at Lock 14 would raise the water to Elevation 352, giving an effective head of about 57 ft. Lock 12 is 40 miles above Wetumpka, and here the water level is raised to Elevation 420, making the effective head of 67 ft. These sites are all shown on the general map, Fig. 2.

The proposed development on the Tallapoosa River is at Cherokee Bluffs, about 30 miles northeast of Montgomery. Here a dam 180 ft. high would back the water up 30 miles in the river, making one of the largest storage reservoirs in the country, the area being 34 000 acres.

The Little River development is in the northeastern part of the State (see Fig. 2), about 33 miles northeast of Gadsden. The devel-

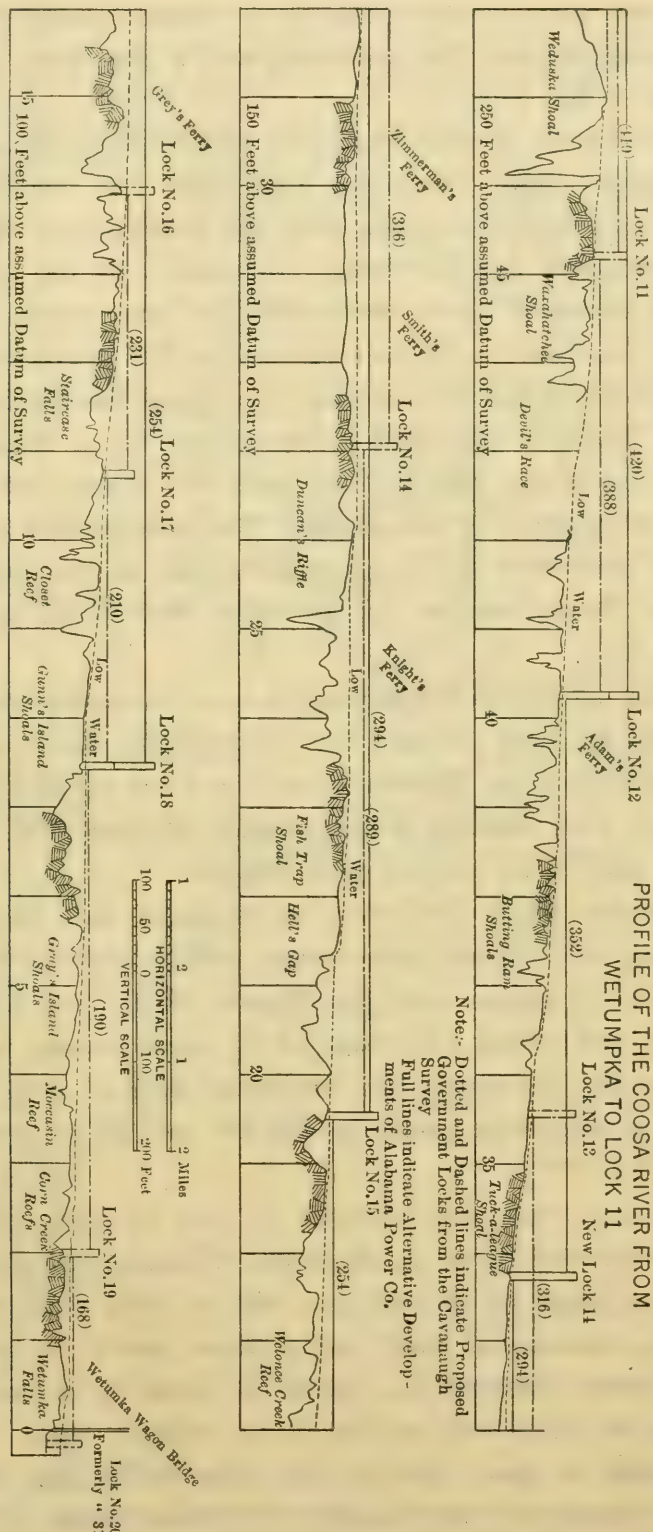

PROFILE OF THE COOSA RIVER FROM
WETUMPKA TO LOCK 11
Note:- Dotted and Dashed lines indicate Proposed Government Locks from the Cavanaugh Survey
Full lines indicate Alternative Developments of Alabama Power Co.
HORIZONTAL SCALE
VERTICAL SCALE
2 Miles
200 Feet
250 Feet above assumed Datum of Survey
150 Feet above assumed Datum of Survey
100 Feet above assumed Datum of Survey
Lock No.11
Lock No.12
Lock No.13
New Lock 14
Lock No.14
Lock No.15
Lock No.16
Lock No.17
Lock No.18
Lock No.19
Lock No.20
Formerly " 31
Weduska Shoal
Wauxahatchee Shoal
Devil's Race
Low Water
Adam's Ferry
Butting Ram Shoals
Tuck-a-league Shoal
Zimmerman's Ferry
Smith's Ferry
Duncan's Riffle
Knight's Ferry
Fish Trap Shoal
Hell's Gap
Welonee Creek Reef
Grey's Ferry
Staircase Falls
Closet Reef
Gunn's Island Shoals
Gray's Island Shoals
Moccasin Reef
Corn Creek Reefs
Wetumka Falls
Wetumka Wagon Bridge
(420)
(388)
(352)
(316)
(294)
(289)
(254)
(231)
(210)
(190)
(168)

FIG. 1.

opment at this point would consist of a storage reservoir on Little River, a tunnel through the mountain, and a pipe line to a power-house in the valley below.

The developments on the Tennessee River are at Muscle Shoals, a short distance above Florence. Two plans for the development of the water-power in connection with the improvement of the river have been proposed, one involving two dams and the other three, each scheme, however, would accomplish the same result.

Possible Combination of Developments.—The Government engineers have proposed the construction of a large storage reservoir on the Etowah River, at the head-waters of the Coosa River, for the purpose of regulating the flow of both the Coosa and Alabama Rivers, in the interests of navigation. The construction of this reservoir would obviously be of great advantage to the developments on the Coosa River, because the low-water flow would be increased to 4 700 sec-ft.

Considering the Coosa River projects as a connected system, tying together Lock 12, Lock 18, and the intermediate developments, and using the storage of the Etowah Reservoir, there would be produced in the dryest year 143 000 of continuous, 24-hour, 80% efficiency, horse-power, and, in an average year, 268 000.

The Cherokee Bluffs Reservoir, as proposed, will have an area of 34 000 acres, and the available storage will be 50 000 000 000 cu. ft. Using this in connection with the Muscle Shoals and Little River developments, and taking advantage of its great storage in times of low water on the Tennessee, the continuous 24-hour, 80% efficiency, horse-power in dry years would be 334 000, and in an average year 505 000.

By combining these two systems, it will be seen that, in the ultimate development, there is obtainable 24-hour, 80% efficiency, to an amount in excess of 550 000 h.p., with a 100% load factor. On the ordinary commercial load factor there would be required a capacity of about 1 000 000 h.p., which is probably in excess of the amount that the State will use for many years to come.

A knowledge of these facts is only desirable in that they partly explain the reasons for the adoption of certain types of construction as being part of an extensive proposed system.

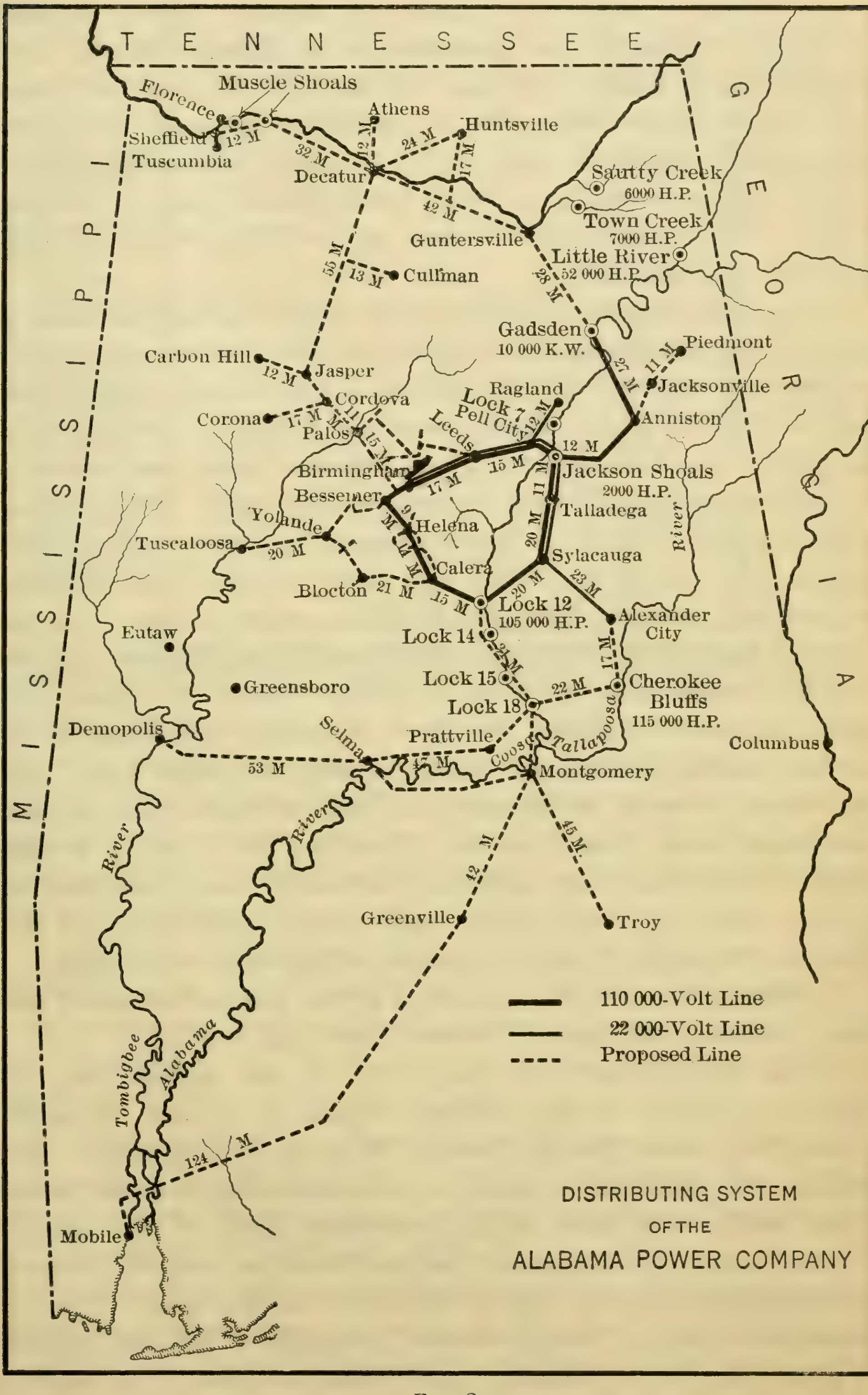

TENNESSEE
MISSISSIPPI
GEORGIA
Muscle Shoals
Florence
Sheffield
Tuscumbia
Athens
Huntsville
Decatur
Guntersville
Cullman
Scutty Creek
6000 H.P.
Town Creek
7000 H.P.
Little River
52 000 H.P.
Gadsden
10 000 K.W.
Piedmont
Jacksonville
Anniston
Carbon Hill
Jasper
Cordova
Corona
Palos
Ragland
Lock 7
Pell City
Leeds
Birmingham
Bessemer
Jackson Shoals
2000 H.P.
Talladega
Yolande
Helena
Tuscaloosa
Calera
Sylacauga
Blocton
Lock 12
105 000 H.P.
Alexander City
Lock 14
Lock 15
Lock 18
Cherokee Bluffs
115 000 H.P.
Eutaw
Greensboro
Demopolis
Selma
Prattville
Montgomery
Columbus
Greenville
Troy
Mobile
Tombigbee
Alabama
River
Coosa
Tallapoosa
110 000-Volt Line
22 000-Volt Line
Proposed Line
DISTRIBUTING SYSTEM
OF THE
ALABAMA POWER COMPANY

FIG. 2.

On Fig. 2 is shown the primary transmission system proposed to connect ultimately the developments at Cherokee Bluffs, Lock 18, and Muscle Shoals, with Lock 12 and with the principal industrial centers of the State. As at present constructed, the system uses power developed at Lock 12, at a steam turbo-generating station at Gadsden, and at a small hydro-electric plant at Jackson Shoals. The cities to which power is now being served are indicated by the solid lines.

Power Market.—With regard to the power market, Alabama at present may be considered as divided into two distinct sections. In the northern half are the coal and ore mines, several cement mills, and all the industries usually found in a mineral section. Birmingham is in this section, its rapid growth being due mostly to the mineral resources of the Birmingham District. Great development is going on there, the present demand for power being greater than in the rest of the State, and there is little doubt that this demand will increase tremendously in the years to come. In the southern half of the State are the cotton-growing districts and the shipping centers along the Gulf of Mexico. Aside from street railways and electric lighting, the power consuming industries of the southern section are limited almost entirely to cotton mills. For this reason, it would be necessary, in order to market the immense quantity of power from Cherokee Bluffs and the Lock 18 developments, to introduce new industries, preferably those dependent on a large supply of cheap power, such as the manufacture of fertilizers of the variety using the fixation of atmospheric nitrogen.

In Table 1 is given the population of the principal cities of Alabama, showing their growth between 1890 and 1910.

Initial Development.—In July, 1912, it was decided that the initial development of the Company should be at Lock 12 on the Coosa River, rather than at Cherokee Bluffs on the Tallapoosa. This decision was based on a number of considerations, principal among which was the fact that in the Government grant of the Lock 12 site the right to develop power was contingent on the completion of the construction of the dam at this point on March 4th, 1914. If construction had not been prosecuted promptly at this site, a grant of great value to the Company would have been lost. The requisite funds for the initial development had been raised, and each day's

TABLE 1.—POPULATION OF CITIES IN ALABAMA.

	Cities.	Population in Census of: 1890.	1900.	1910.
Northern Alabama.	Tuscumbia	2 491	2 348	3 324
	Florence	6 012	6 478	6 689
	Sheffield	2 731	3 333	4 865
	Decatur	2 765	3 114	4 228
	New Decatur	3 565	4 437	6 118
	Gadsden	2 901	4 282	10 557
	Alabama City	.....	2 276	4 313
	Attalla	1 254	1 692	2 513
	Anniston	9 998	9 695	12 794
	Huntsville	7 995	8 068	7 611
	Birmingham	26 178	38 415	*132 685
	Bessemer	4 544	6 358	10 864
	Tuscaloosa	4 215	5 094	8 407
	Talladega	2 063	5 056	5 854
Southern Alabama.	Montgomery	21 883	30 346	38 136
	Prattville	724	1 929	2 222
	Selma	7 622	8 703	13 649
	Greenville	2 806	3 162	3 377
	Andalusia	270	551	2 480
	Mobile	31 076	38 469	51 521

* In 1909 Birmingham absorbed a number of adjoining towns into the "Greater Birmingham." This accounts for the great increase in population; the rate of increase, however, is remarkably large, the population in 1914 being reported unofficially as 166 000.

delay was costing a large sum of money, and furthermore, it would have been impossible, on account of certain legal difficulties, to start construction at Cherokee Bluffs for a number of months. Besides these facts, the original outlay for the Cherokee Bluffs development was estimated to be more than 50% greater, and the cost per installed horse-power somewhat more, than at Lock 12.

A description of the Coosa River, and of the climatic conditions prevailing in the district, will be given, and, as a large proportion of the membership of this Society is resident in and about New York City, comparisons are made with conditions prevailing in that vicinity.

Coosa River.—The Coosa River, which constitutes part of the drainage system of the Mobile Basin, is formed near Rome, Ga., by the confluence of the Etowah and Oostanaula, which have their sources in the mountains of Northwestern Georgia.

The river flows in a general westerly direction over the Piedmont Plateau from Rome into the State of Alabama, where it turns in a southwesterly direction and meanders through wide valleys to Gads-

den. Here, again, the river changes its general trend, turning more toward the south and flowing in a more direct line, between mountains of limestone formations. A short distance below the crossing of the Louisville and Nashville Railroad about opposite Sylacauga, the river passes out of the limestone formations into the slates and schists, and the bed of the stream becomes hard and very rough. From this point the river flows in a southeasterly direction over a succession of shoals and riffles to Wetumpka, where it joins the Tallapoosa River to form the Alabama. In this last section, where the fall in the river is more abrupt, a large quantity of power can be developed economically.

The first project for improving the river for navigation was proposed in 1875, and some work has been done in the general scheme of making the river navigable for light-draft boats. The system, when completed, will be a waterway, almost 900 miles long, from the head-waters of the Coosa to the Gulf of Mexico. Prior to 1912, however, no improvements had been made in the rough section of the river between Gadsden and Wetumpka.

In its upper reaches the Coosa flows through clay-covered territory where it accumulates from its feeders, in all except extreme low-water seasons, a quantity of silt, making it appear very muddy. Tests, however, have shown that this extremely muddy appearance is due in great part to material which has gone into solution in the water, as at its worst it carries in suspension not more than 1 part of silt in 6 000. It is not anticipated, therefore, that there will be any considerable difficulty from silting of reservoirs on this river.

The source of the river is in a region which has a heavier annual rainfall than any section of the United States, with the exception of some parts of the State of Washington. The surface rocks are mostly limestones, which, of course, are only moderately porous; but they are broken up with large fissures, and the solvent action of the rainfall has converted many of these fissures into subterranean channels, as shown by the number of large springs which abound in this territory. To a certain extent, this condition, has a regulating effect on the watercourses emptying into the Coosa, yet in the winter and early spring of the normal year the rains are intense, and the river is subject to floods from December until April, with occasional flashy floods before and after that period, the month of greatest flow being March. The regulating effect of these springs and the fact that in

the average year the rainfall is distributed somewhat uniformly over the months are responsible for the fact that the low-water flow of the Coosa is ordinarily nearly three times as great as that of the Hudson at Albany, and is equal to the low-water discharge of the Mississippi at St. Paul. Table 2 gives the discharge record of the river from 1897 to 1914, inclusive.

The greatest flood known in the river occurred in the spring of 1886, before gauge readings had been commenced. At Lock 12 this flood has been variously estimated, by engineers who have studied and reported on the Coosa River projects, to have been from 95 000 to 170 000 sec-ft. Extensive studies made during 1913 have indicated that the probable flood intensity was approximately 135 000 sec-ft. The average spring flood in the last 14 years, during which time observations have been taken continuously has amounted to 80 000 sec-ft.

The area of the water-shed above Lock 12 is 9 087 sq. miles. Using the estimated maximum flood of 1886, 135 000 sec-ft., this would give a maximum discharge of 14.8 sec-ft. per sq. mile of water-shed, which is one of the smallest in the country. This is explained principally by the fact that the water-shed is very large, and the average intensity of rainfall during a storm is very small. To illustrate this, a few examples of flood discharges are given in Table 3.

The lowest flow known to have occurred was in 1904, at which time observations were taken at Riverside, 72 miles above Lock 12. A comparison of the gauges and rating curves at these two points indicates that the minimum flow was 1 700 sec-ft. at Lock 12. The average minimum flow since observations have been taken amounts to 5 000 sec-ft. With the regulation effected by the construction of the dam at Lock 12, the minimum flow of 1904 would have amounted to 2 400 sec-ft.

Climatic Conditions.—Few people in the North realize that Birmingham, Ala., has had a greater range of temperature, both at the upper and lower limits, than New York City. The records of the United States Weather Bureau show that the maximum temperature of New York City is 100° and that the minimum is — 6°, and that the temperature has ranged between 104 and — 10° at Birmingham.

In summer the heat is intense, but of an entirely different character from that of New York City. For long periods the temperature during the day ranges between 90° and 100°, but heat prostrations

TABLE 2.—DISCHARGE OF THE COOSA RIVER

Year.	January.		February.		March.		April.		May.		June.	
	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
1897 to 1913, inclusive.	19 081	94 150	29 478	106 200	35 447	125 700	25 771	112 500	14 184	71 500	13 107	85 500
1897	10 800	30 800	25 100	42 250	55 000	91 000	23 500	66 500	9 150	13 300	6 500	7 250
1898	15 500	55 000	8 400	22 800	8 600	20 200	25 100	56 000	7 200	13 000	5 800	8 200
1899	14 000	22 300	46 750	83 250	61 500	107 250	35 000	67 750	10 300	17 750	7 250	10 300
1900	17 200	49 750	32 500	91 000	38 300	72 000	43 700	112 500	10 800	19 250	30 500	85 500
1901	36 600	94 150	29 650	65 300	28 000	95 700	45 300	84 800	21 150	64 550	15 800	36 900
1902	32 900	94 150	34 600	85 500	53 000	106 500	28 250	92 500	8 750	11 000	6 750	7 400
1903	14 200	26 300	62 150	106 200	57 600	89 800	37 250	79 000	13 350	40 000	18 500	50 200
1904	7 750	18 250	11 000	23 200	14 250	27 750	10 000	18 700	6 400	8 400	6 300	9 650
1905	18 300	73 100	39 000	87 100	14 700	26 800	9 800	11 750	18 500	53 750	8 750	13 000
1906	30 300	64 600	11 200	20 600	50 500	125 700	19 000	56 000	9 300	14 150	13 150	46 000
1907	20 200	70 000	26 000	61 500	26 150	80 800	13 700	24 100	20 650	57 600	14 000	40 750
1908	23 700	53 000	43 200	83 500	27 000	74 650	20 500	37 250	13 300	18 700	8 500	12 200
1909	19 250	46 750	43 750	81 750	70 000	124 000	25 150	63 800	28 300	68 500	33 750	77 800
1910	10 800	21 750	15 850	49 100	15 000	40 800	8 500	17 750	24 000	71 500	15 900	28 750
1911	17 200	67 000	14 150	29 400	9 750	16 800	33 200	84 750	8 750	13 200	6 250	7 250
1912	16 400	40 024	28 350	52 380	37 800	59 620	34 380	61 770	17 050	39 172	12 020	29 800
1913	27 258	89 000	40 958	64 500	60 845	82 560	22 802	55 700	9 772	17 260	8 237	12 280

are practically unknown. The evenings are cool, and the breezes which blow during the night are refreshing after the heat of the day. In its effect on the speed of the work, and consequently its cost, this heat is to be reckoned with, because labor automatically adapts itself to the climate.

TABLE 3.—FLOOD DISCHARGE OF SEVERAL RIVERS.

Stream.	At	Water-shed, in square miles.	Cubic feet per second per square mile.
Six-Mile Creek	Ithaca, N. Y.	47.5	170
Gallinas River	Las Vegas, N. Mex.	90.0	129
Tohickon Creek	Mt. Pleasant, Pa	102.0	112
Nashua River	Massachusetts	109.0	104
Mora River	La Cueva, N. Mex.	159.0	140
Croton River	Croton Dam, N. Y.	339.0	74
Broad River	Carlton, Ga.	762.0	38
Raritan River	Bound Brook, N. J.	879.0	59
Mohawk River	Little Falls, N. Y.	1 306.0	22
Tallapoosa River	Milstead, Ala.	3 840.0	18
Hudson River	Mechanicsville, N. Y.	4 500.0	15
Kanawha River	Charlestown, W. Va.	8 900.0	13
Coosa River	Lock 12, Ala.	9 087.0	15
Tennessee River	Chattanooga, Tenn.	21 418.0	21
Susquehanna River	Harrisburg, Pa.	24 030.0	19
Mississippi River	St. Paul, Minn.	36 085.0	20
Kansas River	Lecompton, Kans.	58 550.0	4

The significance of the low limit of temperature reached at Birmingham is not as serious as the records would seem to indicate. Periods of low temperature are very rare, and have never been of

AT LOCK 12, FROM 1897 TO 1913, INCLUSIVE.

July.		August.		September.		October.		November.		December.	
Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.
11 991	64 500	10 312	72 800	8 128	62 250	8 895	95 000	7 944	72 400	14 130	96 400
9 250	35 250	6 300	8 400	5 200	6 100	5 050	6 250	5 000	5 550	8 750	17 500
6 800	13 400	12 000	26 800	16 300	62 250	24 000	95 000	10 300	28 700	9 400	13 400
7 800	18 750	6 400	13 200	6 200	9 000	5 450	5 800	5 800	9 650	13 750	36 900
17 500	62 250	7 600	16 250	9 000	31 800	9 500	31 800	10 300	43 800	19 000	30 600
9 000	13 800	27 500	72 800	13 000	31 800	8 150	13 350	6 700	7 150	25 250	96 400
6 250	9 200	6 100	7 900	6 500	13 300	7 000	13 300	6 600	13 000	16 300	33 800
9 950	26 250	8 000	17 500	5 500	7 000	5 600	6 000	5 500	6 700	5 400	5 800
6 000	8 400	8 600	24 250	5 100	5 600	4 500	4 800	4 950	5 100	7 650	18 750
10 700	24 600	9 300	18 700	6 000	7 900	6 600	10 000	5 750	5 800	28 750	56 800
28 700	72 400	14 500	22 900	13 550	19 200	27 200	69 250	23 200	72 400	15 900	46 000
8 450	11 750	7 500	9 650	8 800	29 300	6 700	13 300	12 200	44 500	46 800	38 000
7 500	15 000	7 800	12 150	6 300	13 750	5 600	9 500	5 700	6 300	20 000	63 750
13 750	28 750	15 000	50 500	7 150	11 400	6 750	18 700	5 750	5 800	10 000	20 700
27 000	64 500	8 800	17 750	7 500	15 400	6 000	7 190	5 500	5 600	7 790	24 500
8 550	15 900	9 300	19 200	5 300	5 600	6 700	19 200	8 000	19 650	16 250	56 000
14 650	29 250	10 290	32 000	8 657	16 100	7 524	13 300	5 860	7 320	11 136	19 600
6 752	12 700	6 760	10 500	4 403	17 400	5 635	21 600	3 315	4 000	6 774	14 100

long duration. Ice is never formed to any thickness in ponds or rivers, and the mid-day sun almost invariably causes the temperature of the coldest days to rise above freezing.

It is not necessary, therefore, to consider the effect of an ice thrust in the design of a dam in this climate, and, for the same reason, no precaution need be taken in the design of forebays, for the care and handling of ice at penstock screens. Needless to say, no difficulties are encountered with frazil.

To make clearer the rainfall conditions, Table 4 shows a comparison between the records of normal monthly precipitation at New York City and at Birmingham. From this it will be seen that the rainfall in New York City is more evenly distributed throughout the year than in Birmingham, and that, from December to March, inclusive, the Birmingham rainfall is far in excess of the mean monthly. In July and August, the precipitation at each place is about equal and is the result of heavy showers of short duration.

In the normal year, construction work in Alabama is hampered to a large extent in winter by rains of several days' duration. At such times it is impossible to carry on work, because laborers, even if provided with suitable clothing, will not turn out.

Work Done to Date.—In April, 1912, E. A. Yates, Assoc. M. Am. Soc. C. E., was appointed Chief Engineer of the Company, and an en-

gineering and construction force was organized. A party was put in the field at Cherokee Bluffs for the purpose of surveying the flooded area (34 000 acres) of that reservoir. A short time later an additional party was placed in this reservoir, and a large party was put to work on the surveys for the Coosa River reservoirs. The surveys for the Muscle Shoals reservoirs were made in 1913.

TABLE 4.—COMPARISON OF RECORDS OF NORMAL MONTHLY PRECIPITATION AT NEW YORK CITY AND AT BIRMINGHAM, ALA.

Month.	New York City.	Birmingham, Ala.
January	3.79	5.32
February	3.74	4.75
March	4.10	5.76
April	3.30	3.67
May	3.18	3.09
June	3.26	3.88
July	4.54	4.70
August	4.53	4.48
September	3.59	3.50
October	3.71	2.34
November	3.44	3.39
December	3.45	4.60
Year	44.63	49.48
Mean Monthly	3.72	4.12

At the time the decision had been reached that the initial development should be at Lock 12, no plans of the development had been drawn up, with the exception of a general plan and elevation of the proposed structure. Only 1½ years remained for the construction of the dam within the time stipulated in the Government grant, and, therefore, it was impossible to draw up complete plans before proceeding with the construction. It was desirable to let a contract for the construction of the power-house foundations and dam to a contractor having plant and equipment ready to be moved to the work without delay. Such a contract was let, for the construction of the power-house foundations and dam, on a cost-plus-fixed-fee basis, and the direction of the work, approval of selection of plant, and methods of attack were reserved to the Chief Engineer of the Company. It was decided that the construction of the superstructure and the equipment of the power-house should be done by the Company's own force. As soon as this contract was signed, the necessary preparations for starting construction were inaugurated.

An old lumber railroad (shown on Fig. 3), extended through the hills from a point where it had physical connection with the Louisville and Nashville Railroad, about 5 miles south of Calera, to a point about 5 miles from the dam site. It was known as the Clear Creek Railroad. It was not ideal for construction purposes, because, in its length of 18 miles, it had forty-two timber trestles, one switch-back, and many sharp curves, the sharpest being 35° on a 2.5% grade. The shortest, alternate line, however, would have been at least 14 miles long, and would have involved construction from beginning to end, with a large amount of heavy work. It was necessary to transport the heavy plant to the dam site at the earliest possible moment, and time was not available for the construction of such a line,

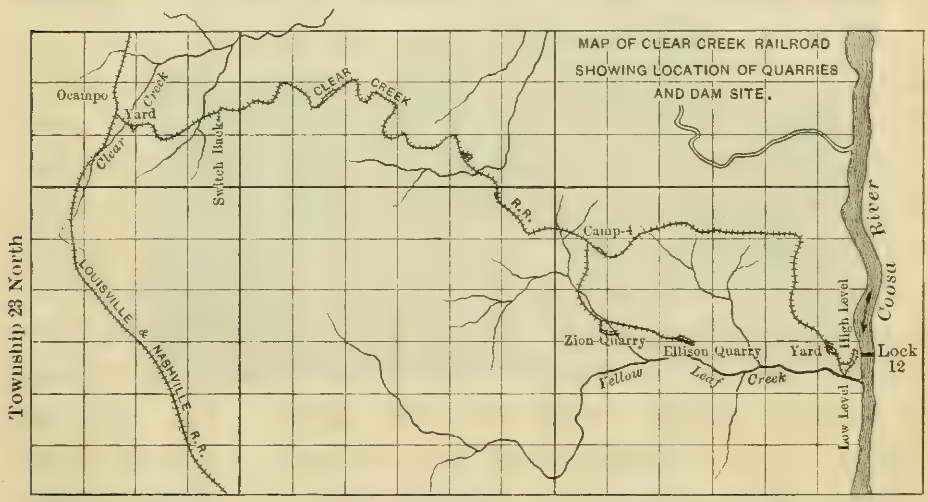


FIG. 3.

although the cost of the railroad as a unit would undoubtedly have been less. The construction and repair work on this railroad was started on August 12th, 1912, by the Company's forces, and on November 26th, the first car loads of the contractor's heavy plant were delivered at the dam site, at least 2 months earlier than would have been possible if an entirely new road had been built. Before this time, however, camps had been constructed at the dam, and preparations were being made for the installation of the plant. During the period when the plant was being set up, a coffer-dam, enclosing the area to be excavated for the power-house and a portion of the spillway, was constructed. This coffer-dam was unwatered for the first time on December 13th, 1912, and the excavation of the river bed was started.

Long delays occurred, due to heavy rains and floods, and it was not until April 12th that the first concrete was placed. In November, 1913, the power-house foundations having been prepared, the building of the superstructure and the installation of machinery were begun. All the concrete work of the power-house foundations and dam was completed on February 26th, 1914, 6 days before the limiting date set by the Government grant. The installation of the first unit was completed on March 22d, 1914. The completion of the installation of the other units followed within a short time thereafter. The first unit was tested out and dried and was put into commercial operation on April 12th, 1914, just 1 year after the first concrete was poured.

An auxiliary steam plant, to be used in building up a load for the Lock 12 development prior to its completion and then as a standby in times of low water, was erected in the first half of 1913.

The Alabama Power Development Company, one of the companies acquired in 1912, had completed the small hydro-electric plant at Jackson Shoals and the transmission line from Anniston to Gadsden. This company also owned a site on the Coosa River, about 2 miles outside of Gadsden, where it proposed to construct a 10 000-kw., steam turbo-generating station. The design of this station had been practically completed by Messrs. Sargent and Lundy, of Chicago, and a contract for the turbines had been signed. Aside from the grading of a spur railroad to the site, no construction work had been done. It was decided in October, 1912, that the Company would build and equip the plant with its own forces, and work was started under a modified design on November 1st, 1912. The plant was completed in June, 1913, but a delay in the delivery of the transformers prevented commercial delivery of power until July. Fig. 4 shows the exterior of the plant and Fig. 6 shows a section through the boiler and turbine rooms.

The construction of transmission lines and sub-stations was carried on simultaneously with the other work. The surveys for proposed transmission lines were started with three parties in July, 1912, the purchasing of right of way was begun in October, and actual construction was commenced in November. In March, 1914, the lines from Lock 12 to Birmingham (46.5 miles), from Lock 12 to Anniston, *via* Jackson Shoals (47.8 miles), from Sylacauga to Alexander City (25.1 miles), from Jackson Shoals to Leeds (26.5

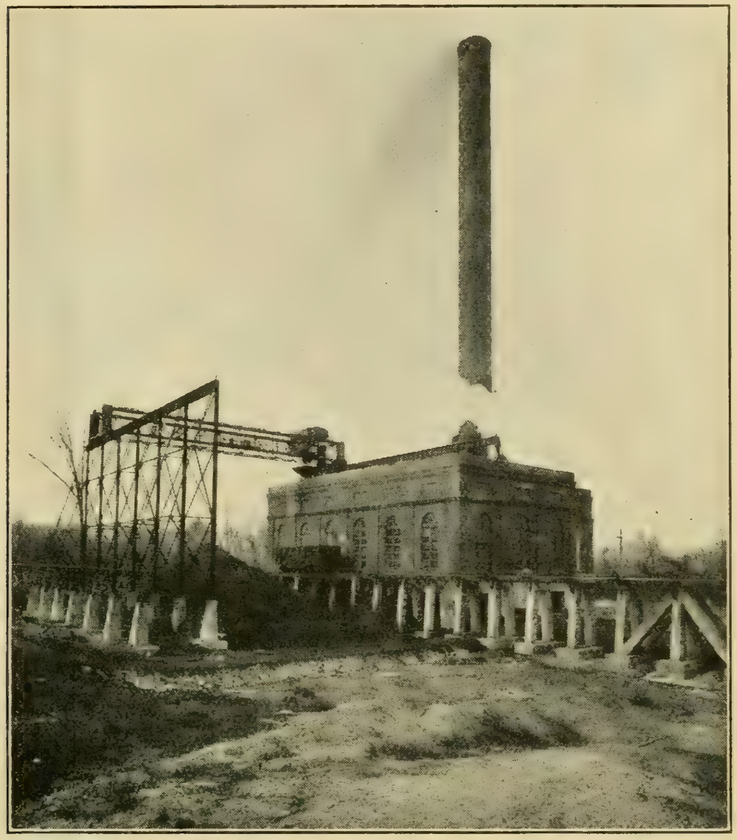


Fig. 4.—Gadsden Steam Plant.

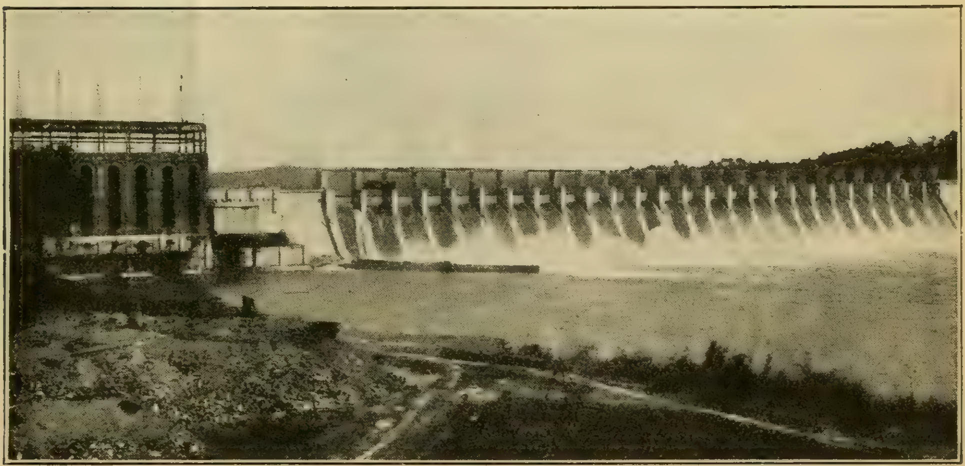
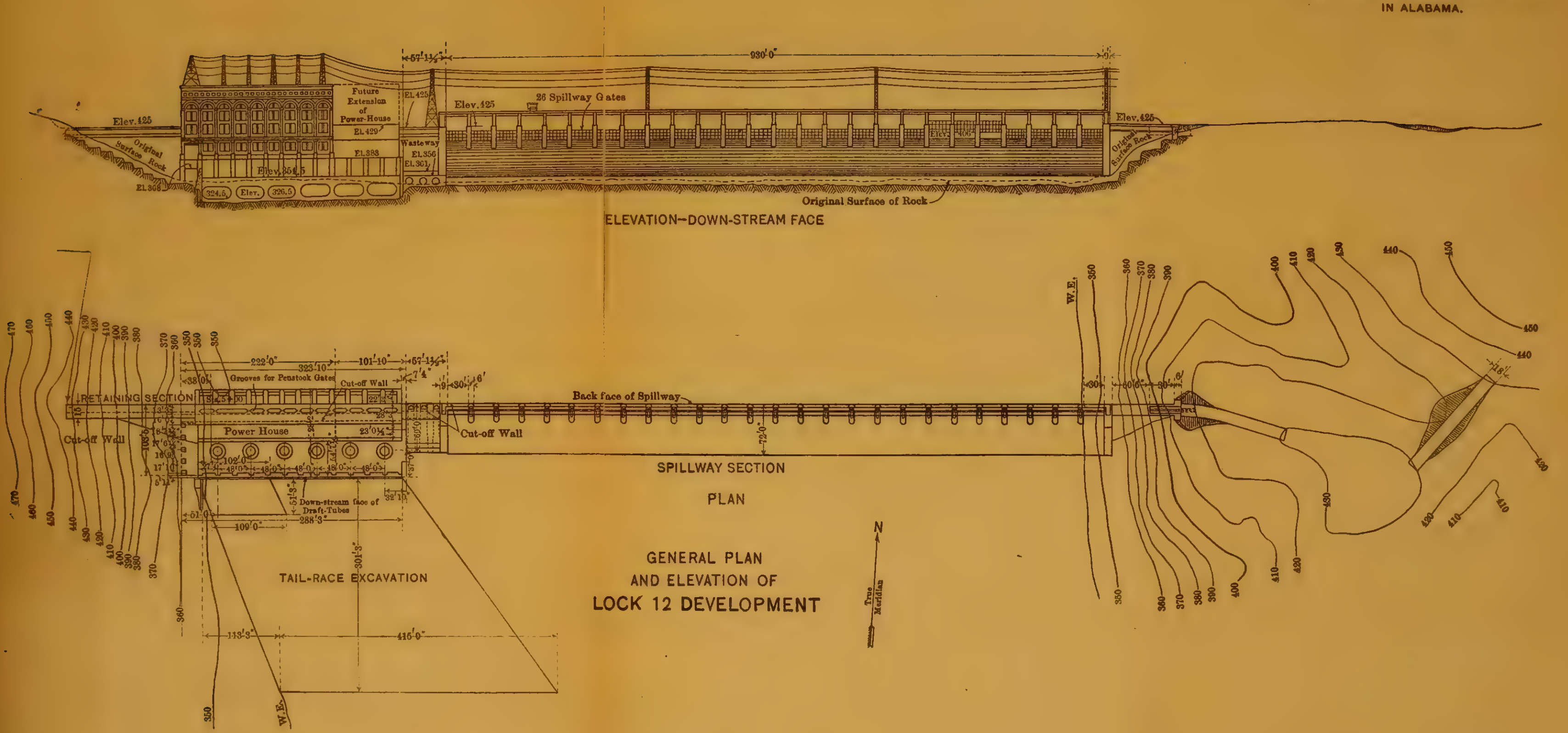


Fig. 5.—Power-House Under Construction, Brickwork Having Reached High-Tension Floor.



GENERAL PLAN
AND ELEVATION OF
LOCK 12 DEVELOPMENT

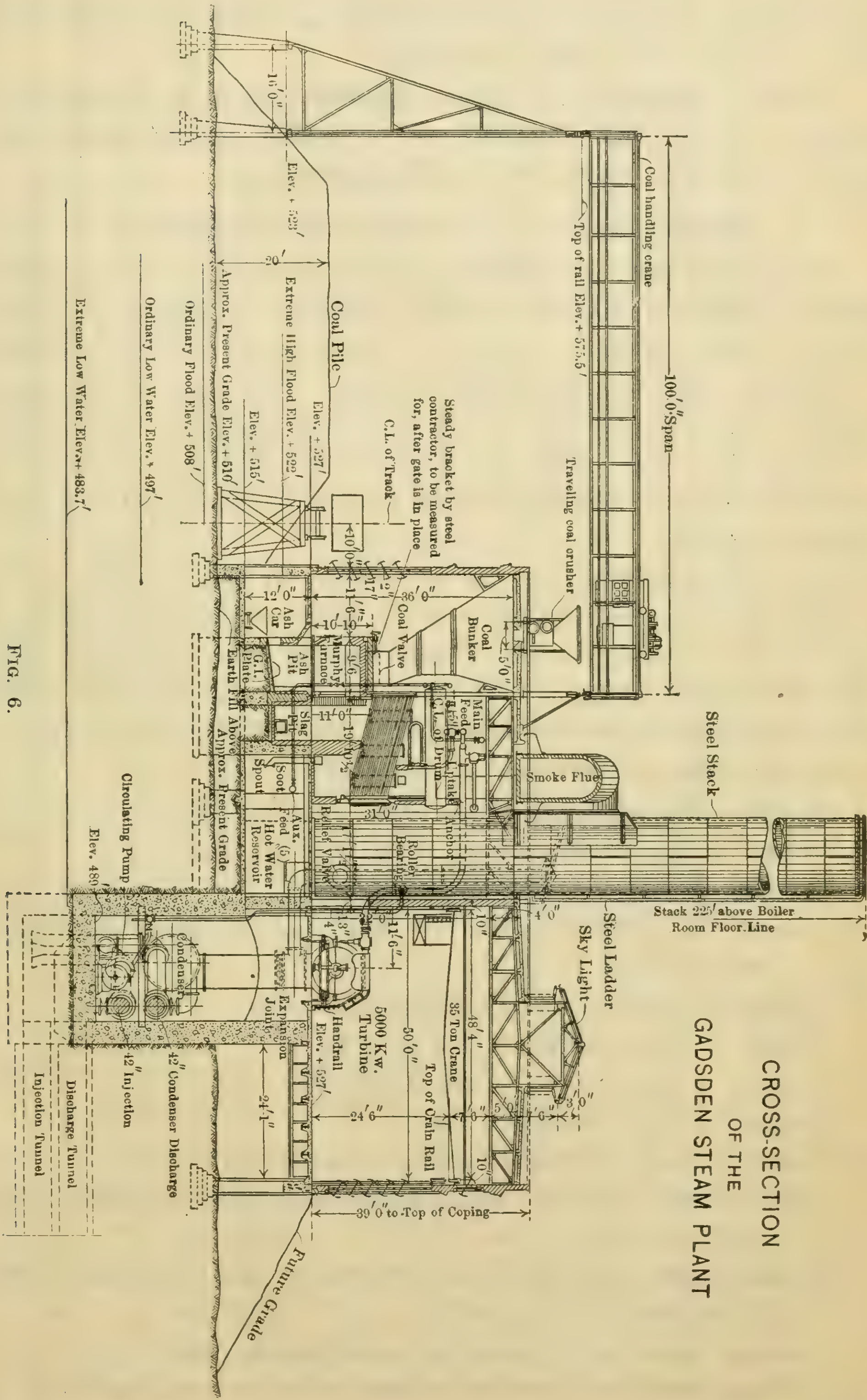

CROSS-SECTION
OF THE
GADSDEN STEAM PLANT
Coal handling crane
Top of rail Elev. + 575.5′
100′0″ Span
Traveling coal crusher
Steel Stack
Smoke Flue
Stack 225′ above Boiler Room Floor Line
Steel Ladder
Sky Light
35 Ton Crane
Top of Crain Rail
5000 Kw. Turbine
Handrail
Elev. + 527′
Expansion Joint
Condenser
Circulating Pump
42″ Injection
42″ Condenser Discharge
Discharge Tunnel
Injection Tunnel
Future Grade
39′0″ to Top of Coping
Coal Pile
Coal Bunker
Coal Valve
Murphy Furnace
Ash Pit
Ash Car
Slag Pit
Soot Spout
Hot Water Reservoir
Roller Bearing
Relief Valve
Main Feed
Anchor
C.L. of Drum
C.L. of Track
Steady bracket by steel contractor, to be measured for, after gate is in place
Earth Fill Above Approx. Present Grade
Extreme High Flood Elev. + 522′
Approx. Present Grade Elev. + 516′
Ordinary Flood Elev. + 508′
Ordinary Low Water Elev. + 497′
Extreme Low Water Elev. + 483.7′
Elev. + 523′
Elev. + 527′
Elev. + 515′
Elev. 480

FIG. 6.

miles), from Leeds to Lovick (5 miles), and 5 of the total of 19 miles of line from Leeds to Birmingham, had been completed, making a total of 155.9 miles constructed.

Outdoor sub-stations of large capacity were built at Anniston, Jackson Shoals, and Gadsden, and a similar though larger substation is under construction at Birmingham.

General Design of the Lock 12 Development.

The site of the dam known as Lock 12 is on the Coosa River, between Chilton and Coosa Counties, Alabama, in Section 24, Township 23 North, Range 15 East, 11 miles on an air line northeast of Clanton, the seat of Chilton County. The site of the locks proper is around the east end of the dam, and their construction at some later date will not interfere in any way with it as now built, being an entirely separate matter, and connected in no way physically with it.

The dam determined on for the purpose at this site was of the straight overflow gravity type, to be built of cyclopean masonry, with a power-house, having foundations of mass masonry, forming its western extremity. The general plan and elevation of the dam and power-house are shown on Plate XXIII. The main dimensions of the structure are as follows:

Length of power-house substructure, over all, 303 ft. 4 in.

Width of power-house substructure, over all, 136 ft. 5 in.

Length of wasteway section, 57 ft. 1½ in.

Length of spillway section of dam, not including end piers, 930 ft. 0 in.

Width of spillway section, at base, 72 ft. 0 in.

Length of west abutment, 134 ft. 6 in.

Length of west core-wall beyond abutment, 34 ft. 6 in.

Length of east abutment, 105 ft. 6 in.

Length of east core-wall, beyond abutment, 53 ft. 0 in.

Total length over all—dam, power-house and abutment—1 530 ft. 5½ in.

Total length over all—dam, power-house and core-walls—1 617 ft. 1½ in.

Average elevation of bottom of cut-off trench, 335.0.

Average elevation of bottom of excavation for dam, 341.0.

Average elevation of bottom of river before excavation for dam, 343.0.

Elevation of crest of dam, 406.0.

Elevation of top of spillway piers, 444.0.

The spillway, forming practically all of the dam east of the power-house, has a net crest length of 780 ft., divided into 26 sections, each 30 ft. long, which are separated by piers 6 ft. long on the axis of the dam, the piers serving to carry flood-gates of the modified Stoney type.

At a point 11½ miles above Lock 12, the Louisville and Nashville Railroad crosses the Coosa on a through bridge of several trusses, and Peckerwood Creek, a tributary stream, immediately above this point, on a through plate-girder bridge. The elevation of the low steel on the truss bridge is 425.3 and of the plate girders 423.5. These elevations, allowing for the back-water effect which is exaggerated to some extent during floods by the Narrows (shown on Plate XXIV) about 5 miles below the dam, fix the normal elevation of the pond at 420, as established by the U. S. Army Engineers.

For the purpose of maintaining the normal pond level during floods, twenty-six flood-gates, each 14 ft. high, were designed to be placed on the crest of the spillway at Elevation 406. The abutment sections of the dam and the section between the power-house and the spillway in which are the wasteway culverts, are carried to Elevation 425, with parapet walls to Elevation 429.5.

The power-house is shown in cross-section by Fig. 7, on which it should be noted that the foundations are of mass concrete, the penstocks, scroll casings, and draft-tubes being moulded directly in the concrete with no lining of any kind.

One feature of this development, which is a decided improvement over most of those heretofore constructed, is that the units are entirely self-contained, that is, the turbine, generator, and exciter are self-supporting, and can be built complete, from the base ring of the turbine to the top of the direct-connected exciter, without requiring any lateral support from the concrete about the turbine casing during construction.

There are four units in the power-house, and provision in the foundations for two more. Each of the units has a capacity of

17 500 h.p. delivered to the shaft at a speed of 100 rev. per min., and under a head of 68 ft.

The adoption of this size of installation was dependent on many factors and a number of assumptions, a detailed statement of which is impossible here. It might be well, however, to note that the principal considerations involved were the ordinary working head, the reduction of head at times of flood, the nature and extent of the power market on which the load factor depends, the available steam auxiliary plants, and the relation of the plant at Lock 12 to the other proposed developments of the Company.

As noted previously, the year of lowest flow on record was 1904, when the discharge of the Coosa at Lock 12 dropped to 1 700 sec-ft. The storage of the reservoir, using a 10-ft. draw-down, increases the available draft to 2 300 sec-ft., which, with a 68-ft. head, will produce 14 200 h.p., of continuous, 24-hour power, having 80% efficiency. Combining this with the daily output of a steam plant with a capacity of 15 000 kw., there would be produced 614 000 kw-hr. per day, which, on a 50% load factor, is equal to 51 000 kw. maximum demand. A study of the horse-power percentage of time curve for Lock 12 shows that 70 000 h.p. on a 50% load factor, which is equal to 35 000 h.p., continuous, is available at Lock 12 for 80% of the time under a head of 68 ft. and 85% of the time under a head of 74 ft.

It should be noted that this is a pioneer development in Alabama, and that the character of the market is not as well defined as it is in most States where development has been going on for years. It was not known whether part of the load would be electrochemical, or not, and it was necessary to assume the load factor on the best information obtainable from a hurried survey of the market.

It is estimated that the construction of the Etowah Reservoir at the head-waters of the Coosa, as previously mentioned, will increase the low-water flow of the river to 4 700 sec-ft. Assuming that Lock 12 would be operated ultimately in connection with this storage reservoir and the developments lower down the river, an ultimate installation of 105 000 h.p. is provided for in the power-house foundations.

PLATE XXIV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.

MAP OF LOCK 12 RESERVOIR

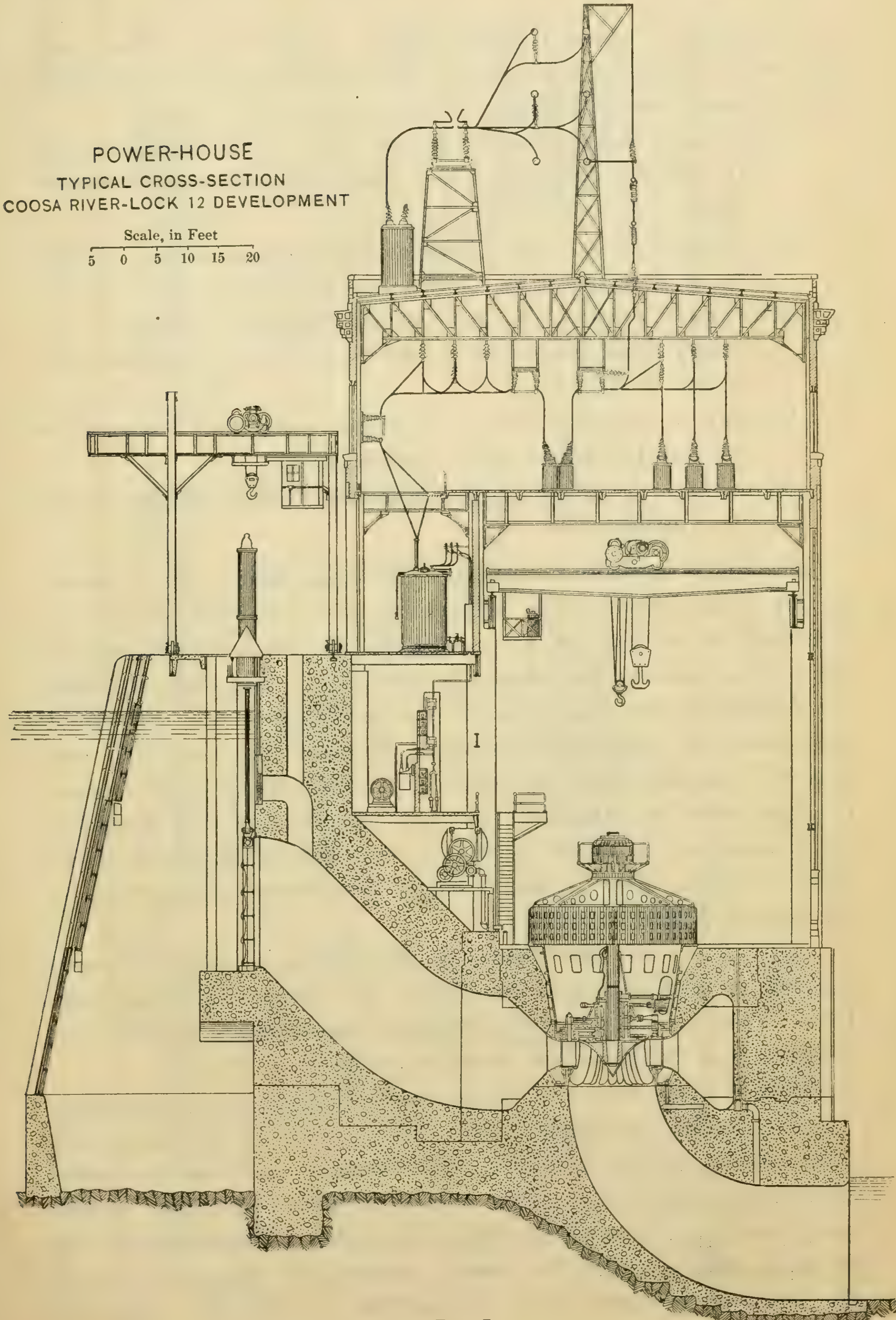

POWER-HOUSE
TYPICAL CROSS-SECTION
COOSA RIVER-LOCK 12 DEVELOPMENT
Scale, in Feet
5 0 5 10 15 20

Fig. 7.

Dam.—Fig. 8 shows the cross-section of both the spillway and the retaining sections of the dam. The assumptions of loading made in the design were as follows:

A maximum depth of water on the crest of the spillway of 19 ft., which is equivalent to a flood of approximately 200 000 sec-ft.;

An elevation of the water, back of the retaining section, of 425 ft., or 5 ft. above the normal lake level;

An upward pressure on the base of the dam varying in intensity from two-thirds of the head at the up-stream face to zero at the toe;

Weights of masonry and of cyclopean stone were assumed at 150 and 160 lb. per cu. ft., respectively;

As previously noted, it was not necessary to allow for any thrust from ice pressure.

The line of pressure developed under these assumptions is shown on the cross-sections. The assumption of an upward pressure of two-thirds at the head of the up-stream face, decreasing to zero at the toe, is probably too severe, in view of the fact that, although the foundation was slate rock, it was quite tight, and the dip of the strata was extremely favorable. However, the fact that this dam will unquestionably be called on to pass floods of 14 ft. over the crest, with the attendant vibration, and the fact that Montgomery, a large part of which is built on low ground, is only 40 miles down the river, made it imperative to use extreme assumptions on the side of conservatism.

Fig. 9 shows the cross-section of the spillway at Lock 12 superimposed on the cross-sections of the La Grange and Big Bend Dams. The former is reported to have passed floods of 15 ft. on the crest, and the latter was designed for floods of 20 ft.

Power-House.—Fig. 7 shows a cross-section of the power-house. The foundations throughout were of mass concrete, reinforced with steel rods around the penstocks, scroll casings, and over the draft-tubes. The computation of the stresses in such a structure is very complicated, as not only the ordinary provisions of dam design must be given attention, but the stresses transmitted from one part of the structure to another must be carefully investigated. The large

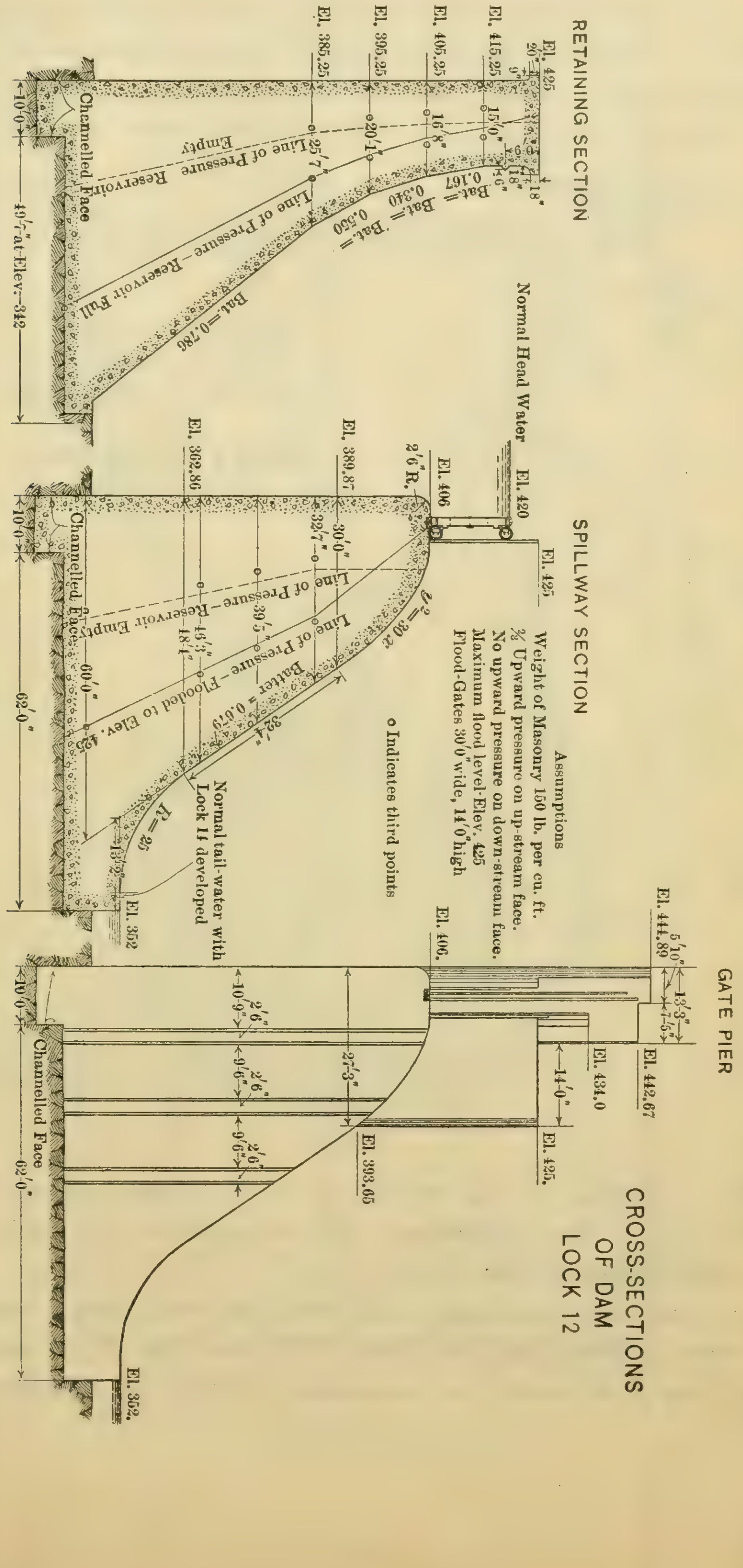

CROSS-SECTIONS
OF DAM
LOCK 12
RETAINING SECTION
SPILLWAY SECTION
GATE PIER
Assumptions
Weight of Masonry 150 lb. per cu. ft.
⅔ Upward pressure on up-stream face.
No upward pressure on down-stream face.
Maximum flood level-Elev. 425
Flood-Gates 30'0" wide, 14'0" high
o Indicates third points
Normal Head Water
Normal tail-water with
Lock 14 developed
Line of Pressure—Reservoir Empty
Line of Pressure—Reservoir Full
Line of Pressure—Flooded to Elev. 425
Channelled Face

FIG. 8.

open spaces within the structure permit of only approximate solutions, and ample safety factors must be allowed to cover the uncertainties. In determining the stability of the structure, the same assumptions as to loading were made as in the case of the retaining section of the dam. No allowance was made for the added

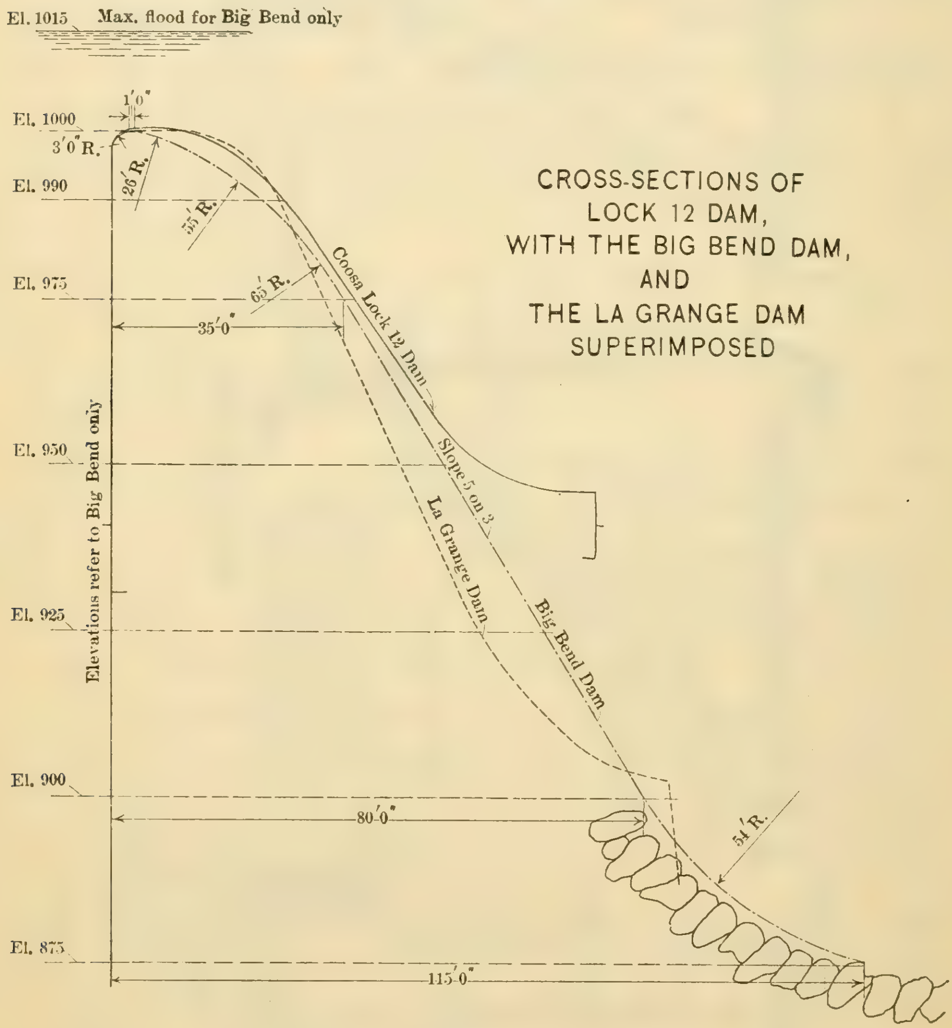


FIG. 9.

stability furnished by the weight of the superstructure and equipment, that is, the foundations were designed to be stable under full water pressure when bare.

As very little experience had been had with wheel casings of the volute or scroll type (see Plate XXV and Fig. 10), it was anticipated that there would be considerable difficulty in moulding them

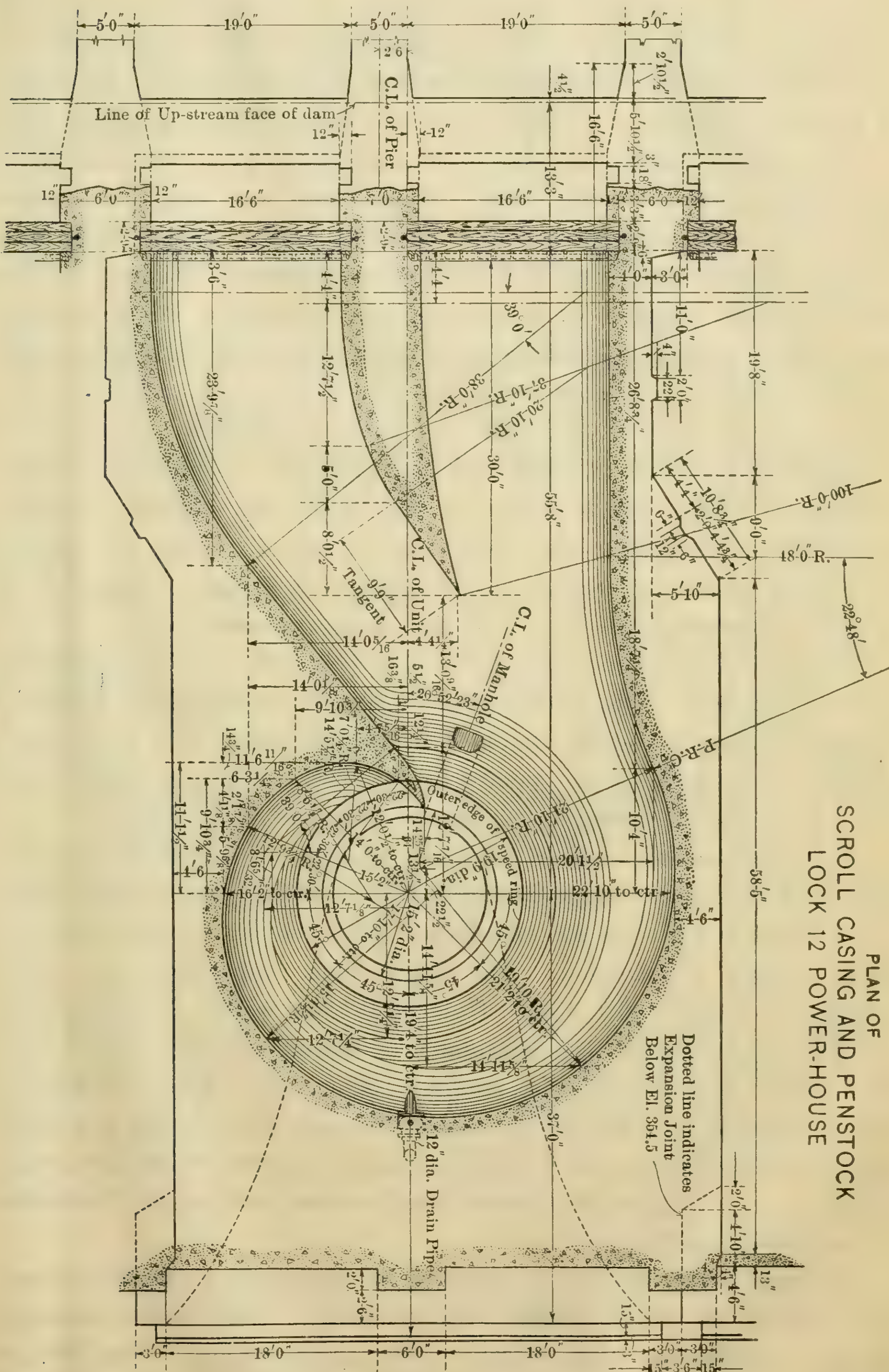

PLAN OF
SCROLL CASING AND PENSTOCK
LOCK 12 POWER-HOUSE
Dotted line indicates
Expansion Joint
Below El. 354.5
Line of Up-stream face of dam
C.L. of Pier
C.L. of Unit
C.L. of Manhole
Tangent
Outer edge of speed ring
12" dia. Drain Pipe

FIG. 10.

directly in the concrete. For this reason, the structure of the forms was given careful study in the designing office and, as a result, no great difficulties were experienced in the form shop or on the work. A detailed description of these forms and their working is given later.

In order to limit the size of the penstock gates to economical dimensions, the penstocks were divided into two for the greater part of their lengths by concrete dividing walls. Vents were provided, running from the roof of the penstocks to the top of the dam, for the purpose of preventing the formation of a vacuum when the penstocks are emptied. For easy access to the upper side of the turbines, a manhole with a ladder between the penstocks and the top of the dam is provided. To assist in facilitating repairs, a manhole, closed with a water-tight door, makes it possible to pass directly from the scroll casing to the draft-tube. When the penstock gates are closed, the invert of the scroll casing is drained by a pipe to the lower end of the draft-tube, which is operated by a valve having a long stem reaching to the generator floor.

To facilitate the opening of the penstock gates, by equalizing the pressure on both sides, provision is made for flooding the penstocks through a by-pass conduit moulded in the concrete. The size of this conduit is such that the penstocks will fill in a reasonable time, even after the turbine-gate clearances increase from wear.

Comparative estimates of alternate designs of reinforced concrete and steel and brick superstructures indicated that, under the conditions existing at Lock 12, the latter would be the cheaper and the more quickly constructed. The power-house superstructure, therefore, has been built with a steel frame and brick walls. A good quality of dark red shale brick was obtainable in the district at a very reasonable price. Concrete blocks, moulded at the site, were selected for the trim work and for the cornice. The use of the dark red brick with whitened joints, and window sashes painted with aluminum paint, in connection with concrete band courses and cornice, furnishes a very pleasing effect.

The main columns of the building, of which there are thirty-four, are built-up of plates, angles, and channels. They were fabricated in two lengths, the splice being at the crane girders. The 100-ton traveling crane is supported on girders, 60 in. deep, carried on the main columns of the building. The roof trusses have a clear span

PLATE XXV.
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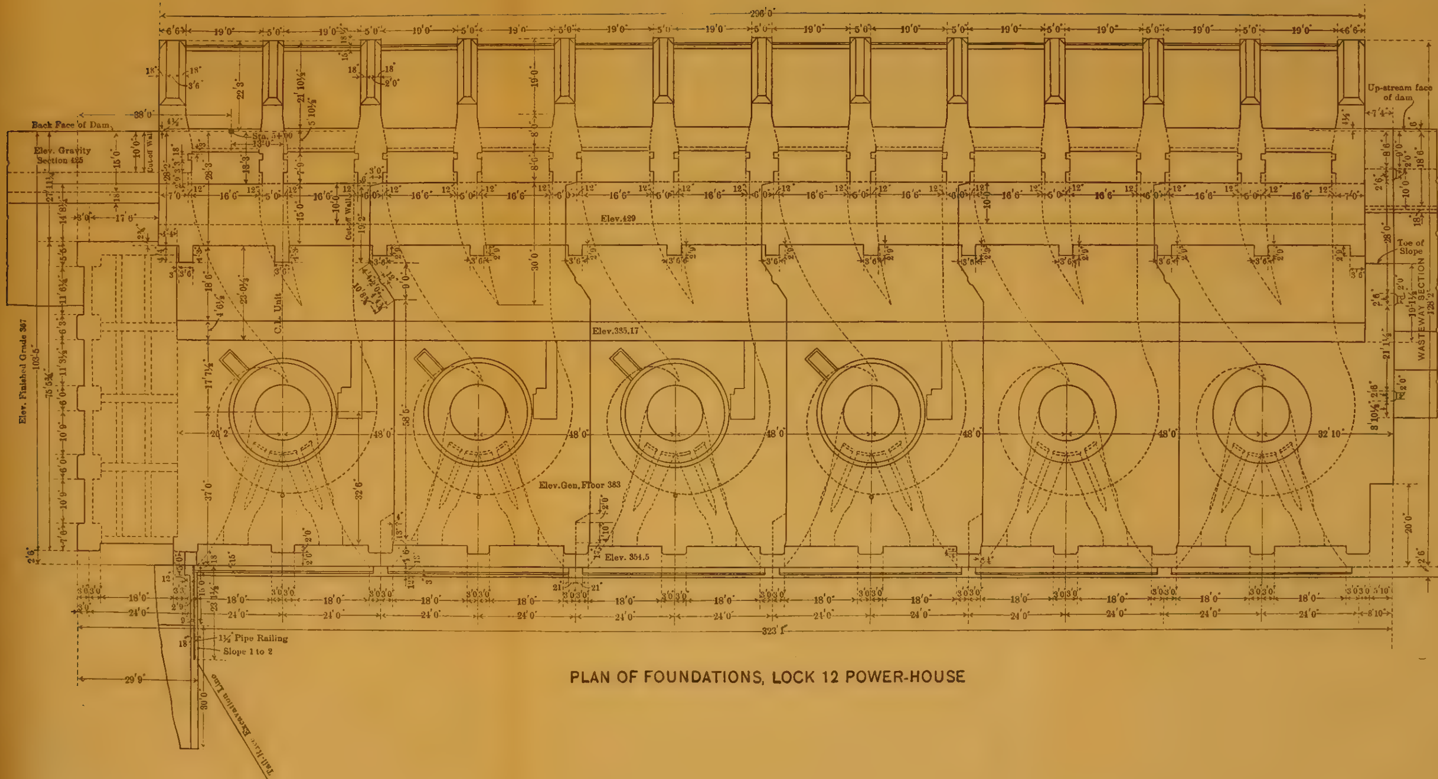


PLAN OF FOUNDATIONS, LOCK 12 POWER-HOUSE

of 69 ft., are 9 ft. deep at the mid-point, and, in addition to supporting themselves and the dead load of the roof, are designed to carry, attached to their lower chords, the high-tension bus structure of the power-house, and, on top of the roof, the electrolytic lightning arresters and several outgoing line towers.

The floors of the power-house superstructure are of reinforced concrete slabs supported on steel beams.

Turbines.—The turbines selected for this installation are the largest capacity, vertical, single-runner turbines ever installed in the United States, to operate under a head of approximately 70 ft.

Fig. 11 is a cross-section through a complete unit, and Figs. 12 and 13 show the runner and the pit liner, respectively. The weight of a runner and shaft complete is 103 000 lb., and the weight of a complete turbine is 520 000 lb.

As previously mentioned, one feature in which these turbines differ from most of those heretofore installed is that they are self-contained. The bottom of the pit liner rests directly on the concrete foundation at the scroll-casing level, where anchorages make it possible to adjust the level of any point on it. This arrangement makes it possible to set up the turbine complete in the shop and do all the necessary machine work there, eliminating the very expensive machine work commonly done in the field. The installation of the turbine on its permanent foundation then becomes a comparatively simple matter, involving only the accurate setting of the base ring on the anchorages and the bolting up of the other sections of the pit liner to this ring. If the machine work in the shop has been accurate, the assembly in the field must be simple. The runner of these units is of cast iron and of the Francis inflow type, cast in one piece. The diameter of the runner, over all, is 13 ft. 3 in., and it is 7 ft. 2 in. high.

The casing or pit liner is made up of three rings; a heavy cast-iron ring, in two sections, forms a foundation for the rings above and transmits the weight of the generator and moving parts from them to the concrete foundation; it also serves the purpose of guiding the water from the scroll casing to the movable guide-vanes. This ring of the pit liner is commonly called the speed ring.

Above the speed ring is the intermediate ring. This serves to transmit the load to the speed ring, to form a water-proof lining for the operating chamber over the runner, and to carry the two gate-

operating engines of the turbine. Above this is the third section of the pit liner, the foundation ring, which rises to the elevation of the main floor of the power-house. On this ring is set the generator armature or stator, and the weight of the latter and of all moving parts is transmitted through it. Bolted to this ring, and forming part of it, is

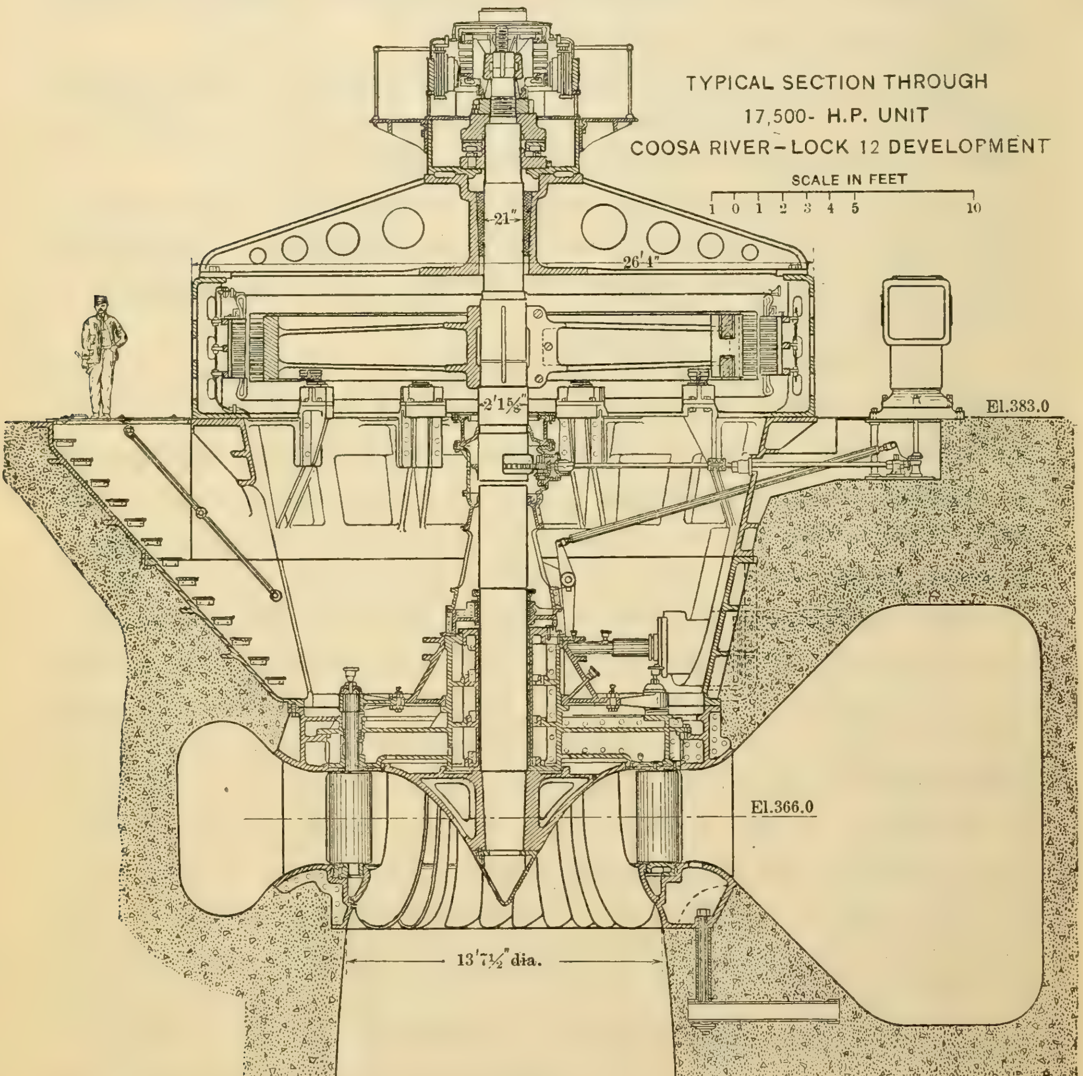


FIG. 11.

the cast-iron stairway giving entry from the generator floor to the operating chamber. The floor of the operating chamber is the cover-head, which is between the speed ring and the intermediate ring of the pit liner.

The shaft, 24 in. in diameter, is one solid piece, passing throughout the turbine, generator, and exciter. The weight of the shaft

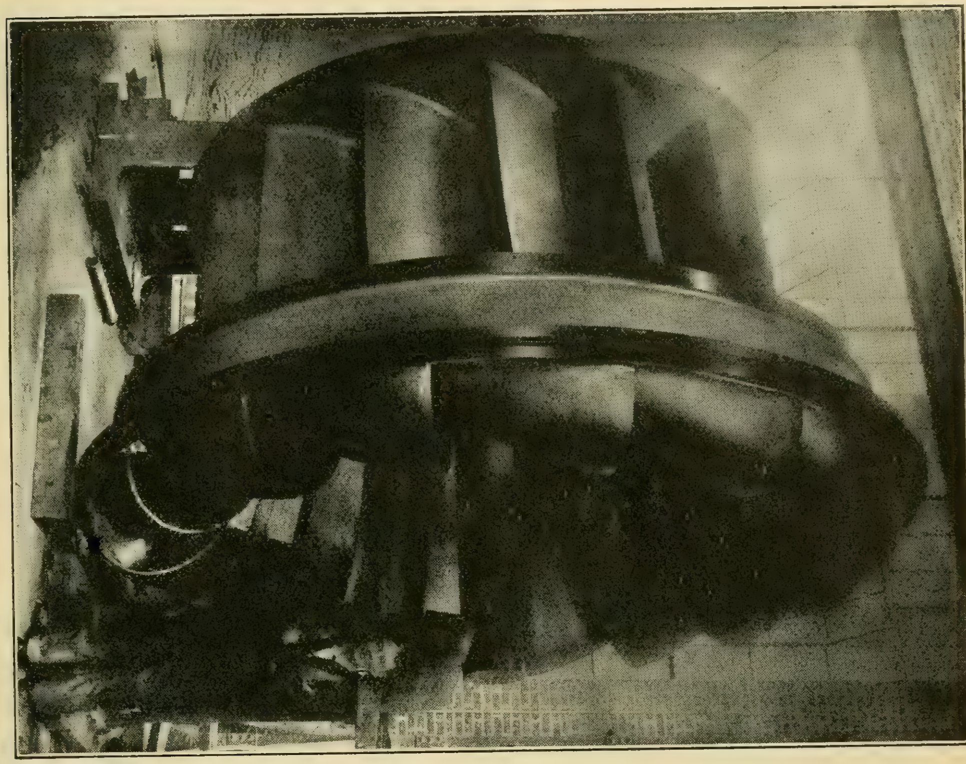


FIG. 12.—RUNNER OF LOCK NO. 12 TURBINES IN SHOP.

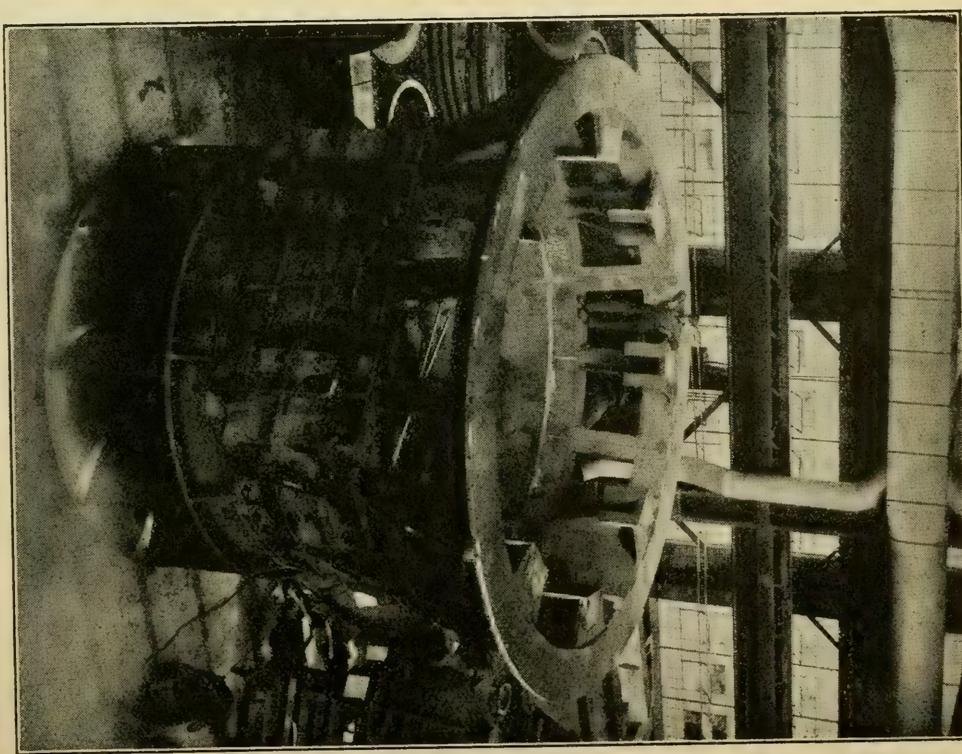


FIG. 13.—COMPLETE PIT LINER OF TURBINE ASSEMBLED IN SHOP.

and the running parts attached to it is transmitted through a thrust-bearing, immediately above the generator and below the exciter, to the bridge on the generator. The runner is made a taper fit on the shaft, the torsion being taken on a long vertical key. A circular key and a cast-iron cap hold the runner in place. The shaft is held in position above the runner by lignum-vitæ guide-bearings in the cover-head of the turbine and by a Babbitted steady bearing under the bridge of the generator. The former is lubricated with water, and the latter with oil. The length of contact of the lignum-vitæ guide-bearings is 70¼ in.

The turbines in the plant are guaranteed by the manufacturers to deliver at the shaft 17 500 h.p., with a head of 68 ft. and a speed of 100 rev. per min.

The runner selected was stepped up from a model which, on a Holyoke test, developed a maximum efficiency of 89.68%, and the manufacturers have guaranteed that the units as installed will have a maximum efficiency of 87% when delivering to the shaft from 15 000 to 17 000 h.p. The efficiencies guaranteed at full and part gate are as follows:

Power.	Efficiency.
17 500 h.p.......	83 per cent.
15 300 "	85½ "
13 125 "	83 "
8 750 "	74 "

The efficiencies obtained in the Holyoke tests are higher than have ever been attained on any turbine heretofore constructed, regardless of speed, capacity, or head.

No opportunity to check the actual efficiencies of the installed units will present itself until the next low-water season, in the summer or fall of 1914, and it is to be hoped that those in charge of the operation of the plant at that time will present a full account of the tests, as they should prove of great interest to the Profession.

These questions received much study, in connection with this development, and it was decided to use vertical units principally because it is possible to obtain higher efficiencies with large capacity vertical than with horizontal units. The reason for this is that in the draft-chests of large horizontal units there are of necessity sharp bends

causing losses of head, and there are other losses from the confluence of streams in the draft-tubes of multi-runner horizontal units. The ideal draft-tube is obtainable with the single-runner, vertical turbine, in which the water is received from the runner at high velocity and conducted away with gradually decreasing velocity to the tail-race, with small loss of effective head from friction or other causes. Volute or scroll casings which, in the last few years, experience has shown to return the highest efficiencies, are impossible to obtain economically in large capacity horizontal units. Besides these considerations, there are other objections to horizontal units in large capacity low-head installations, such as the facts that the unit would be below the level of tail-water, under flood conditions the power-house required would have to be larger, and the structural features of the horizontal shafts for these large wheels would not be satisfactory.

With regard to single *versus* multi-runner vertical units, the following points deserve mention: The ideal scroll casing, by which the inflowing water is brought to the runner at a uniform velocity at all points, cannot be obtained with multi-runner turbines. The effect of multi-runner turbines on draft-tubes is to cause either sharp bends, thereby decreasing the efficiency, or, in an effort to avoid this, require the lengthening of the unit to uneconomical proportions.

The single-runner unit makes possible an ideal arrangement of the operating mechanism, by which it is at all times open to inspection in the operating chamber above the runner. Other advantages of this type are that it is possible to lubricate the gate stems with grease cups, and that there are fewer parts to get out of order.

The selection of speed involves a close study of the inter-relations of power, efficiency, and cost per horse-power developed. In general, other things being equal, the cost of turbines per horse-power developed decreases as the speed increases, the horse-power increases as the speed increases, and the efficiencies at part gate fall off greatly as the speed increases.

An illustration of this last consideration is shown by Table 5, which is a comparison of the efficiencies of these runners at different speeds, under a head of 68 ft. for this installation.

For this development the runner developing 18 500 h.p. at a speed of 100 rev. per min., was selected because a great increase in power was obtainable over the lower-speed runner, with only a small sac-

rifice of efficiency under full load, and the half-load efficiency held up well above that of the higher-speed runner. The consideration that the head would decrease in time of flood to 56 ft. argued against the adoption of a runner with a half-load efficiency as small as 73 per cent. It should be noted that these efficiencies and capacities are not the same as those guaranteed for the full-sized runner, but are those developed from Holyoke tests on model runners.

TABLE 5.

	Efficiency at:		
	94.8 rev. per min.	100 rev. per min.	120 rev. per min.
Developing full power	90.6%	89.5%	90.1%
Developing half power	80.0%	77.0%	73.0%
Full power	16 850 h.p.	18 500 h.p.	17 500 h.p.*

* This runner was smaller than either of the other two, which accounts for the lower power than at 100 rev. per min.

Thrust-Bearings.—The function of the thrust-bearing, as previously noted, is to transmit to the bridge of the generator frame the weight of all the rotating parts and, in addition, the water thrust. Prior to 1912, roller- and step-bearings were the types in general use in hydraulic turbines. As its name signifies, the roller-bearing depended on a nest of rollers to reduce the friction while transmitting heavy loads in motion. In some forms of step-bearings, the load is carried by hydraulic or oil pressure at the foot of the shaft. A combination of these two types is in use in some plants. In 1912 a bearing of an entirely different design, which previously had been introduced into steam turbine practice, was applied to the hydraulic turbine. In October, 1912—when the selection of thrust-bearings was taken under consideration—there were only thirty of these Kingsbury bearings in use, and they were mostly in steam turbines under unit pressures of from 300 to 500 lb. per sq. in. of bearing surface. One was in use at the McCall's Ferry Plant and one additional had been ordered. Fig. 14 shows this bearing.

The lower casting with the pockets is stationary, and is bolted fast to the bridge of the generator frame. The segmental castings, with the flat Babbitted surface and the circular pocket on the underside, are the bearing blocks or shoes. These bearing blocks are sup-

ported on disks, the upper surface of which is a sphere of large radius, the disks fitting into the pockets under the bearing blocks. A collar, securely fastened to the shaft and accurately machined on the lower surface, rests on the bearing blocks when the unit is completely assembled. As the bearing blocks rest on a spherical surface, they are free to turn slightly as if on a pivot. When ready for operation, the interior parts of the bearing are completely immersed in oil. The fundamental principle of operation of the bearing, as expressed clearly by the inventor, is "perfect automatic lubrication effected by the pivotal support of the stationary bearing blocks or 'shoes,' which permits the formation of an oil film, completely separating the sliding surfaces."

This bearing seemed to do away with the objectionable features of the roller-bearing, namely, large friction losses, high initial cost, inaccessibility, and wear of the moving parts.

The bearing placed in June, 1912, in a 10 000-kw. unit at McCall's Ferry, was under a load of 410 000 lb. at 94 rev. per min. (equivalent to a unit pressure of 350 lb. per sq. in. of bearing surface) and was inspected in October, 1912, after 3½ months of steady service. The Babbitted faces of the shoes had been scraped before being placed, and they still showed distinctly the marks of the scraper. The area of contact, as shown by the bright area of the Babbitt, had extended but slightly, showing that the wear had been negligible. A computation made from the rise of temperature of the oil supplied to the bearing indicated a coefficient of friction of 0.0008. The bearing is made accessible for inspection of the Babbitted surfaces by wedges under the shoes, several of which may be removed at once, without lifting the shaft. The bearings, as made for the McCall's Ferry Plant, had an outside diameter of 48 in. The shaft diameter is 21 in.

After a careful consideration of the claims for the Kingsbury bearing and an inspection of that in service at McCall's Ferry, it was decided to adopt it. As constructed, it is 42 in. in outside diameter and supports a load of 330 000 lb.

A duplicate oiling system has been provided for the lubrication of these bearings. There are two tanks, each with a capacity of 838 gal., on the high-tension floor, from which the oil flows by gravity to the casing of the bearing. Provision has been made for the supply of 15 gal. per min. to the bearings, but this quantity will be regulated as

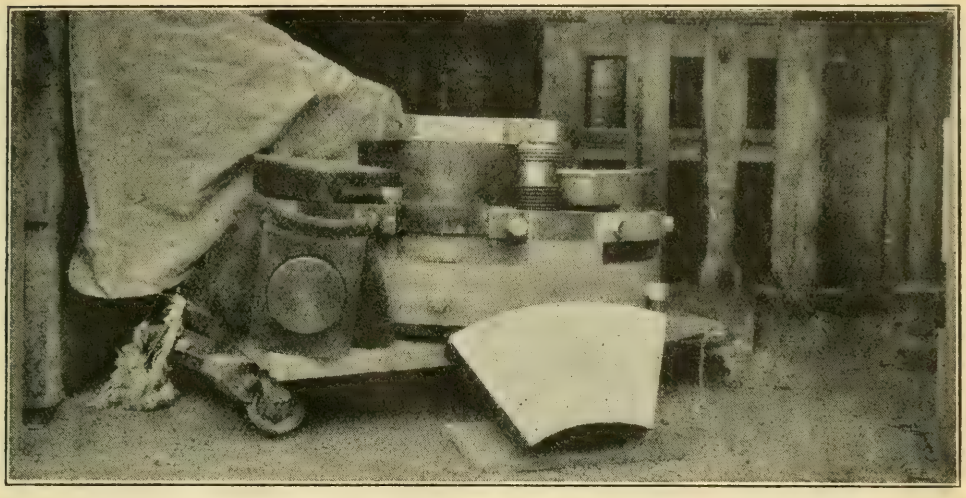


FIG. 14.—KINGSBURY BEARING.

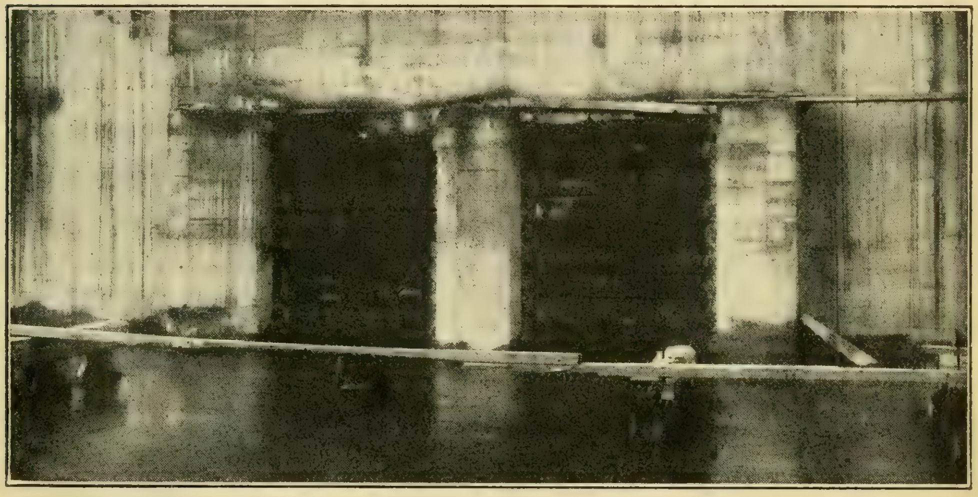


FIG. 15.—TYPE OF TIMBER SLIDE-GATES USED TO CLOSE SEVERAL OF THE STREAM-CONTROL CULVERTS.

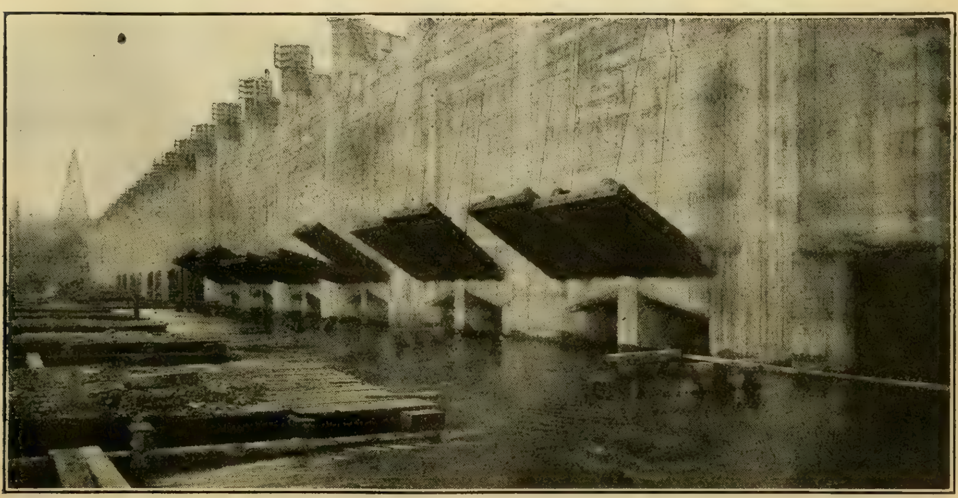


FIG. 16.—STREAM-CONTROL CONDUITS THROUGH DAM, WITH FLAP-GATES IN POSITION FOR CLOSING.

experience develops the best method of working the system. The temperature of the oil at the entrance to and exit from the bearing may be noted with the thermometers provided, so that the supply may be regulated. On leaving the bearing, the oil is pumped back through tanks, in which there are cooling coils, to the tanks on the high-tension floor. These coils are 2 ft. in diameter, are made up of 32 turns of 1½-in. pipe, and have a cooling surface of 105 sq. ft. No experience was available relative to the desirable capacity of these coils; their design is based on judgment alone.

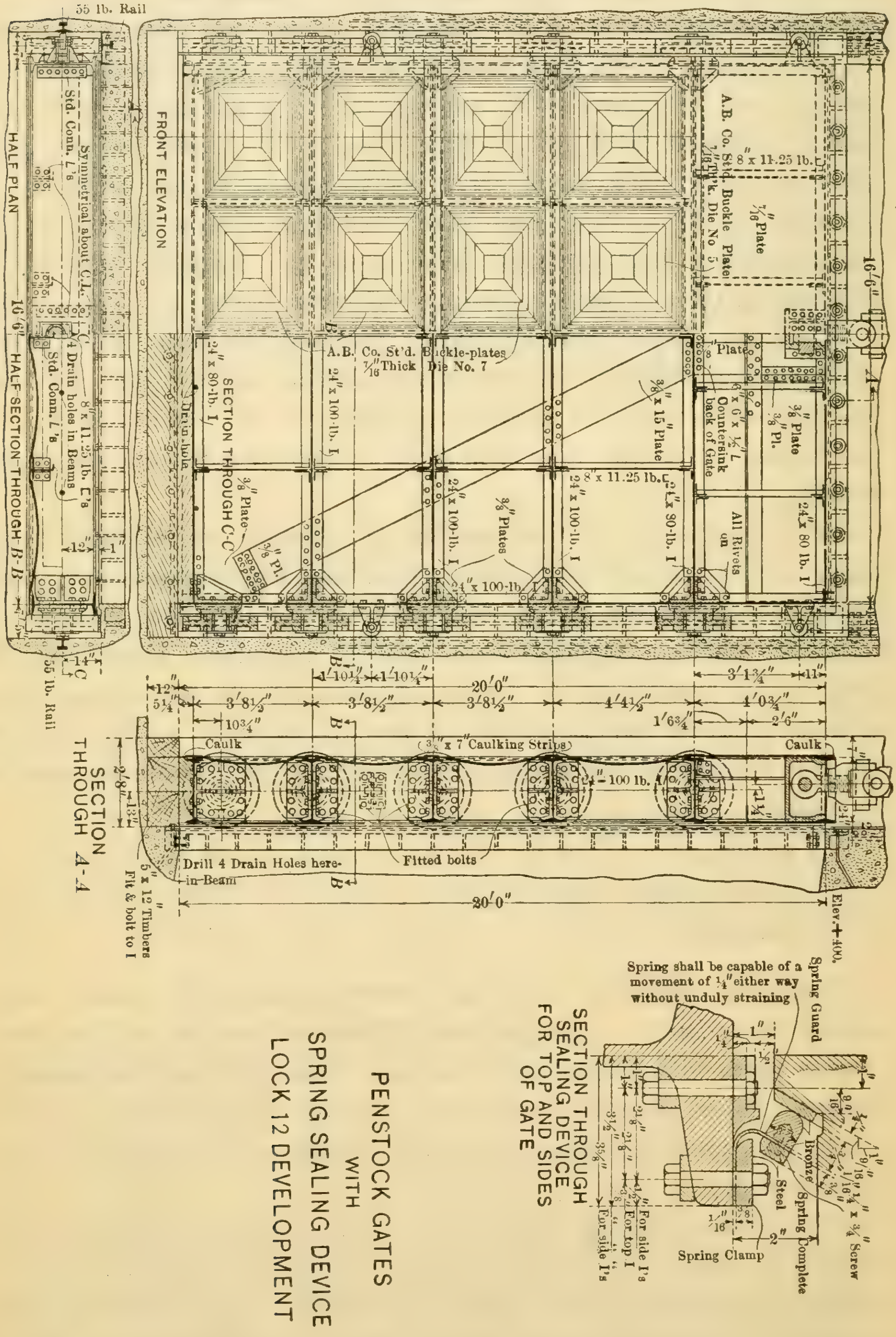
Generators.—The four generators installed are 13 500 kv-a., 6 600-volt, 3-phase, 60-cycle generators, to operate at 100 rev. per min. The generator stator is mounted directly on the foundation ring of the turbine, to which it transmits the load received from the thrust-bearing.

The ventilation of the machine is effected by taking, through ducts in the concrete foundation, fresh air from outside the power-house and circulating it by the fan action of the rotor through the armature and out through holes left in the stator frame into the body of the power-house. The ducts provided for the purpose have a capacity of 60 000 cu. ft. of air per minute at a velocity of 2 000 ft. per min. The efficiencies guaranteed by the manufacturers are as follows:

Full load	95.7 per cent.
Three-quarter load	95.0 " "
One-half "	93.5 " "

Exciters.—Mounted above the thrust-bearing, and securely bolted to the main shaft, is the exciter. These exciters are of the shunt-wound type, with interpoles. The field frame rests on the generator bridge. They are rated at 150 kw., at 100 rev. per min., and 250 volts. An auxiliary motor generator exciter of 150 kw., 250 volts, is provided.

Governors.—The governors are four, modified Keokuk type, actuators, with independent motor-driven pumps, and tanks for the control of the double servo-motors furnished with the turbines. This apparatus is guaranteed, at 200 lb. per sq. in. pressure, to open or close the turbine gates completely in 2 sec., if not called on to develop a greater energy than 250 000 ft-lb. The governors are also



PENSTOCK GATES WITH SPRING SEALING DEVICE LOCK 12 DEVELOPMENT

FIG. 17.

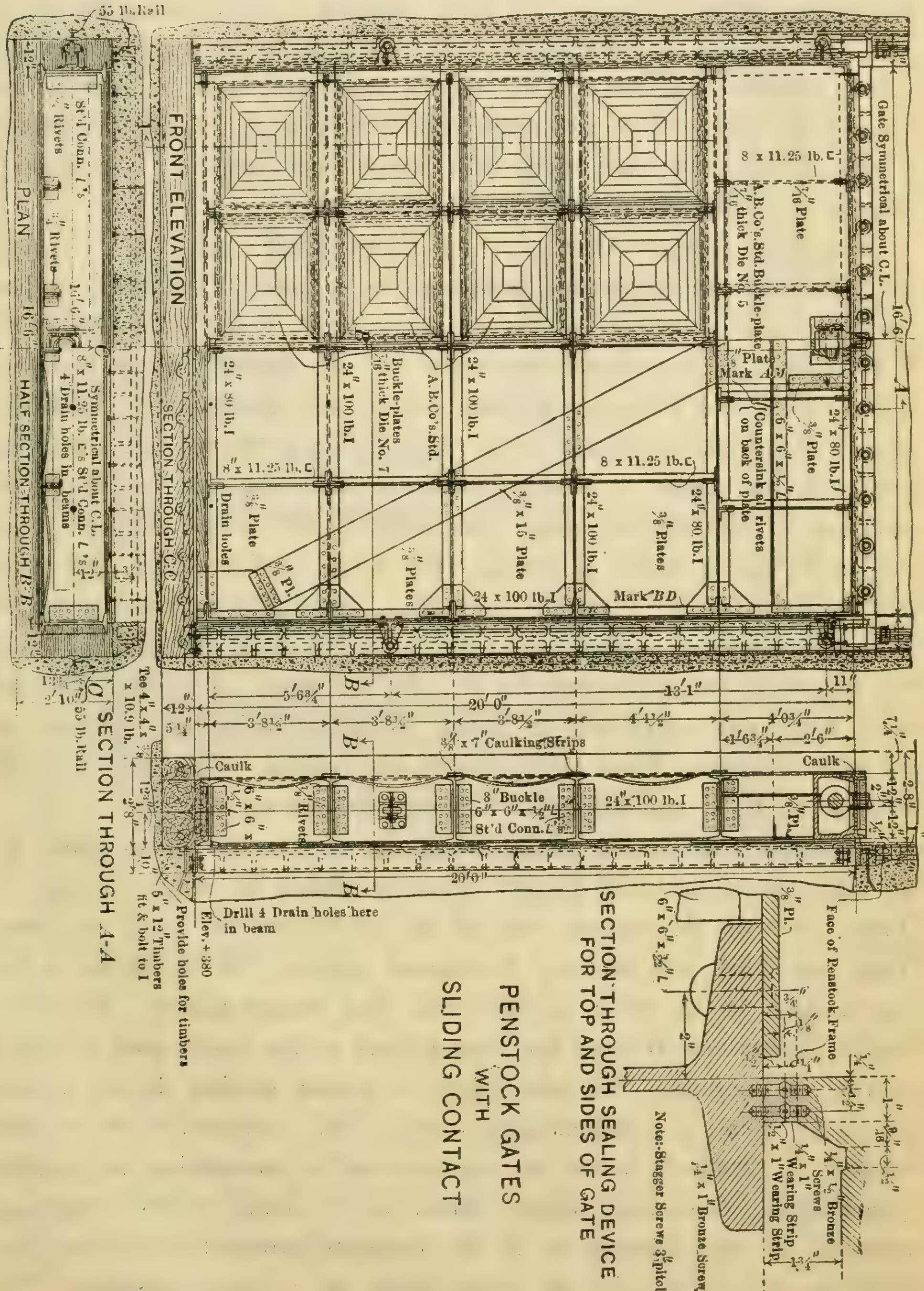

PENSTOCK GATES
WITH
SLIDING CONTACT
FRONT ELEVATION
PLAN
HALF SECTION THROUGH B-B
SECTION THROUGH C-C
SECTION THROUGH A-A
SECTION THROUGH SEALING DEVICE
FOR TOP AND SIDES OF GATE
Gate Symmetrical about C.L.
Note:-Stagger Screws 3" pitch
Face of Penstock Frame
55 lb. Rail
Caulk
3/8" x 7" Caulking Strips
Drill 4 Drain holes here in beam
Provide holes for timbers
Elev. + 380

FIG. 18.

guaranteed to stand substantially steady when the speed does not vary, to be dead beat in action, and not to hunt, to correct with maximum promptness for all load changes within the capacity of the water-wheels to which they are attached, to maintain the speed steady within one-half of 1% under uniform load on the water-wheels to which they are attached, to begin to adjust the water-wheel gates when the speed has varied one-half of 1%, and to operate perfectly in parallel with one another.

Penstock Gates.—As shown on Plate XXV and Fig. 10, each penstock has two entrances, separated by a pier. This made it possible to reduce the size of the gates to 16 ft. 6 in. wide and 20 ft. 0 in. high. The penstock gates are of two types: one is equipped with rollers and a spring sealing device, and the other is a sliding gate with bronze wearing strips. One of each type is used on each penstock, the gate on rollers being opened first to facilitate the opening of the sliding gate. In general, the gates of both types are made up of 24-in. I-beams with $\frac{7}{16}$-in. buckle-plates, and properly stiffened (see Figs. 17 and 18). The gate-lifting apparatus, shown on Fig. 7, consists of a cast-iron cylinder for each gate, 30 in. in inside diameter and 20 ft. $11\frac{1}{2}$ in. long. A piston attached to a 6-in. rod, 24 ft. $6\frac{1}{4}$ in. long, is operated in this cylinder by oil pressure taken from the governor oil system. The cylinder walls are $1\frac{3}{4}$ in. thick, and were tested to 300 lb. per sq. in.

Spillway Gates.—On the spillway, as previously noted, there are 26 gates of a modified Stoney type, as shown by Figs. 19 and 20. Their over-all dimensions are 14 ft. high and 32 ft. 9 in. long. They are built up of four horizontal girders, 36 in. deep at the center and 18 in. deep at the ends, and buckle-plates. The horizontal girders are framed into box-girders at the ends, each of which carries two rollers. The same type of bronze sealing device is used as on the Type A penstock gates, and if this is found to be unsatisfactory in practice, it can be removed and a staunching rod readily substituted. Cast-iron guides, bolted to the ends of the gates and running in cast grooves set in the concrete, prevent the gates from blowing out of line in the wind when in a raised position. The gates are suspended at each end with flat steel ropes passing over drums at the top of the piers. The drums are actuated by shafting and gears, which may be operated from an electric hoist car mounted

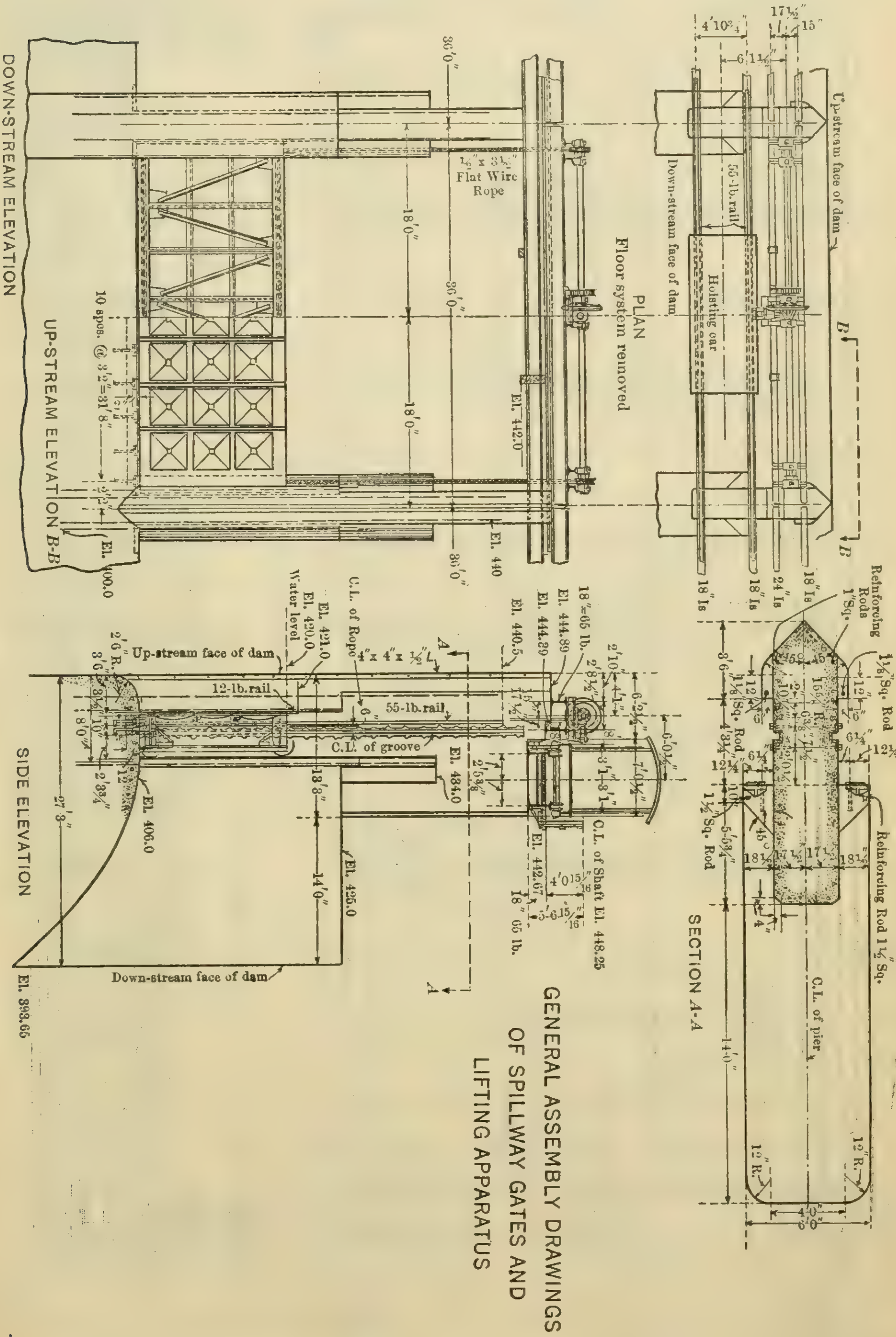

DOWN-STREAM ELEVATION
UP-STREAM ELEVATION B-B
PLAN
Floor system removed
Hoisting car
Up-stream face of dam
Down-stream face of dam
55-lb. rail
Flat Wire Rope
SECTION A-A
Reinforcing Rods
Reinforcing Rod
C.L. of pier
SIDE ELEVATION
12-lb. rail
55-lb. rail
C.L. of groove
C.L. of Rope
C.L. of Shaft El. 448.25
Water level
GENERAL ASSEMBLY DRAWINGS
OF SPILLWAY GATES AND
LIFTING APPARATUS

FIG. 19.

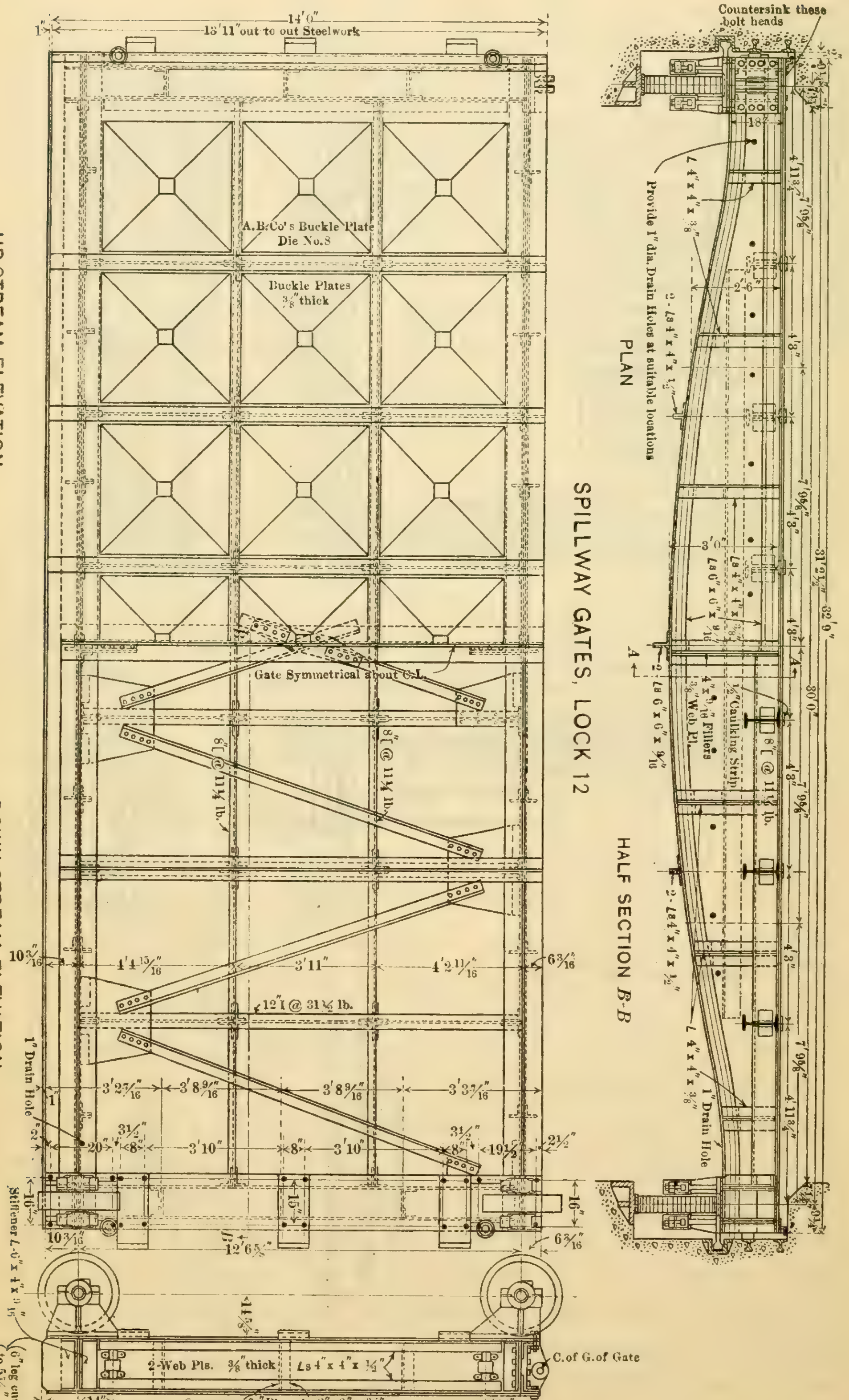

SPILLWAY GATES, LOCK 12
UP-STREAM ELEVATION
DOWN-STREAM ELEVATION
PLAN
HALF SECTION B-B
A.B.Co's Buckle Plate Die No.8
Buckle Plates 3/8" thick
Gate Symmetrical about C.L.
14'0"
13'11" out to out Steelwork
Countersink these bolt heads
Provide 1" dia. Drain Holes at suitable locations
1/2" Caulking Strip
1" Drain Hole
C. of G. of Gate
2-Web Pls. 3/8" thick
Separator 3/8" Pl.

Fig. 20.

on trucks above the piers. As added security, there is a duplicate operating car, equipped with a steam hoist, and, if necessary, both cars may be operated by hand.

Lock 12 Construction.

The construction of the dam and power-house at Lock 12, as laid out and built, is described in detail in the following pages. In considering the various features of this work, the paper is divided into four general headings.

Section A consists of a general outline of the work to be done, its location and description, its general features, and the methods which were planned and adopted for doing it.

Section B covers the methods adopted for handling the work, the materials used, the camps, railroad facilities, description and layout of the contractor's plant at the dam and quarries, the storage and care of materials, and the inspection of all work and materials.

Section C describes in detail the prosecution of the work and the various construction features—the coffer-dams, excavation methods, forms, mixing and placing of concrete, completing the work, and the final closure of the dam.

Section D covers the details of both the contractor's and the Alabama Power Company's organizations for handling the work in the field, their relations, and responsibilities.

Section A—General Outline of Work to Be Done.

Lands.—The land secured, in addition to the actual site of Lock 12, was as follows: On the east bank of the river, a strip averaging 1 800 ft. wide, back from the river bank, extending down stream below the line of the dam 1 400 ft. and up stream about 4 000 ft.; on the west bank of the river, on which all camps, contractor's plant, etc., were located, there was owned, in all, at the immediate site, a strip extending an average distance of 1 900 ft. back from the bank from a point 1 600 ft. down stream to a point 2 400 ft. up stream from the dam. This latter property was used entirely for construction purposes. Considerably more property was owned farther up the basin, but those mentioned are in the immediate vicinity of the dam site.

In addition to the lands described, at or near the site of the dam, on the west bank of the river, the timber rights to 3 250 000 ft. of lumber were purchased, and on the flooded lands purchased on the east bank of the river, there was another 1 000 000 ft., all of which was cut and used, furnishing nearly all the lumber needed for camps and other purposes.

Surveys at the Dam.—On August 5th, 1912, a party was put in the field at Lock 12 for the purpose of establishing the construction lines and elevations preparatory to starting the work. The tops of the 4-in. pipes, used by the U. S. Government for making shot borings on this site, were visible above the water surface, and the average line of these pipes was indicated as the up-stream face of the dam. This line was first permanently established, monumented, and thoroughly referenced, and an original stake, marked 1°, was established as Station 5 + 00 and so used on all surveys and drawings. This was the starting point of all measurements, and the stationing increased to the eastward from here. The up-stream face of the dam was considered as Station 0 + 00 for measurements at right angles, the stationing increasing down stream. In order that the main construction base line should fall inside the construction limits, a line parallel to and 10 ft. south of the face of the dam was run in, and all work was referred to this base line. Concrete monuments, with railroad spikes embedded in them and center-punched on line, were established on this 10-ft. line, at intervals of about 90 ft., for a distance of 1 000 ft. on each end back from the water's edge. On the west bank, to guard further against loss of lines, one of the monuments was carefully referenced. The original elevation left at this point by the Government was permanently established, and all work was based on it. All surveys of overflowed lands were based on this same datum, and deeds written were referred to it as their basis.

A diamond drill outfit was put on the work about August 5th, 1912, and a series of borings was made across the river, on the line of the up-stream face of the dam, in order to verify the original Government data.

Contract for Substructure and Dam.—A contract, under date of August 1st, 1912, was entered into for the construction of the substructures of the dam and power-house, and under it the work was

prosecuted to completion. It was of the cost-plus-a-fixed-fee type, in which, however, the Power Company, through the Engineering Department, had unlimited powers in the direction of the work, and exercised close supervision over all its details. The general requirements of the contract called for completion by December 31st, 1913.

The purchase of all materials was subject to the approval of the Power Company before orders were placed or any contracts made, and payments made under the contract were divided into three general items: (*a*), (*b*), and (*c*).

Item (*a*) covered the contractor's fixed fee;

Item (*b*) covered rental on plant owned by the contractor; and

Item (*c*) covered all expenditures made by the contractor in the prosecution of the work.

About one-half of the large plant was rented from the contractor, at a certain percentage per month, on an agreed valuation, and a list of the items of this plant was attached to and made a part of the contract. The selection of all plant used under Item (*b*) was subject to the approval of the Chief Engineer. Under Item (*c*) were made all expenditures in the prosecution of the work, and they were subject to approval by the Chief Engineer for all that were not the usual running items. The usual camp expenses were covered under a general approval. All plant actually purchased, the operation and maintenance of the railroad, and like items, were included under this item of the contract.

Sources of Materials, and Methods of Securing and Handling Stone, Gravel, and Sand.—Some time before the work was begun at the dam, investigations of all the surrounding country were inaugurated, for the purpose of finding suitable concrete materials for use in the dam. Prospecting parties were put in the field in the early part of August, 1912, to find sand and gravel pits, or stone quarries, as near the dam as possible. These investigations showed that all rock at the dam site, and for several miles up and down stream, would be unsuitable for concrete purposes, being a thin-bedded slate with numerous quartz seams running through it. No sand suitable for concrete was found at all. Gravel, in small quantities only, and in pockets, was found along the river, and such

sand as was found with it contained such high percentages of clay as to render it unfit for use without thorough washing. Early in the work, therefore, it became evident that sand would have to be secured from outside sources.

The only suitable stone in the district was found, after several months of prospecting, near Zion Church, in Sections 17 and 18, Township 23 N., Range 15 E. (see Fig. 3), and its area was more or less restricted. After carefully prospecting this site at Zion Church and another hill about a mile distant, known later as Ellison Quarry, both sites were purchased. All surface indications and drill borings indicated that Ellison Quarry would be the better of the two; later, however, the contrary turned out to be the case. Both sites were about 11 miles, *via* the construction railroad, from the dam, and were the only available sources of coarse aggregate and cyclopean stone found in the vicinity. Both these hills were heavily overburdened, and, after opening up Ellison Quarry and working it for several months, so many clay seams showed throughout the rock that it had to be abandoned. Although there was plenty of suitable rock available at Zion Quarry, it was only possible to work five derricks, and the stone could not be quarried with sufficient rapidity to satisfy the demands. For this reason, gravel and stone were secured from outside pits to fill out the supply. All the crushing operations were carried on at Zion Quarry, and a general view of the quarry and its layout is shown in Fig. 21.

A chemical analysis of the stone furnished from Zion and Ellison Quarries was as follows:

Aluminum and iron oxide...........	3.44	per cent.
Carbonate of lime..................	50.83	" "
Silica	9.00	" "
Carbonate of magnesia.............	36.38	" "
Combined water and organic matter..	0.35	" "

The stone proved to be of excellent quality for concrete purposes, and was far superior to any purchased from outside commercial quarries.

All sand was furnished by contract at a price not to exceed 25 cents per cu. yd., f. o. b. cars, at Jackson's Lake, near Montgomery, Ala. There was a freight charge of approximately 70 cents per cu.

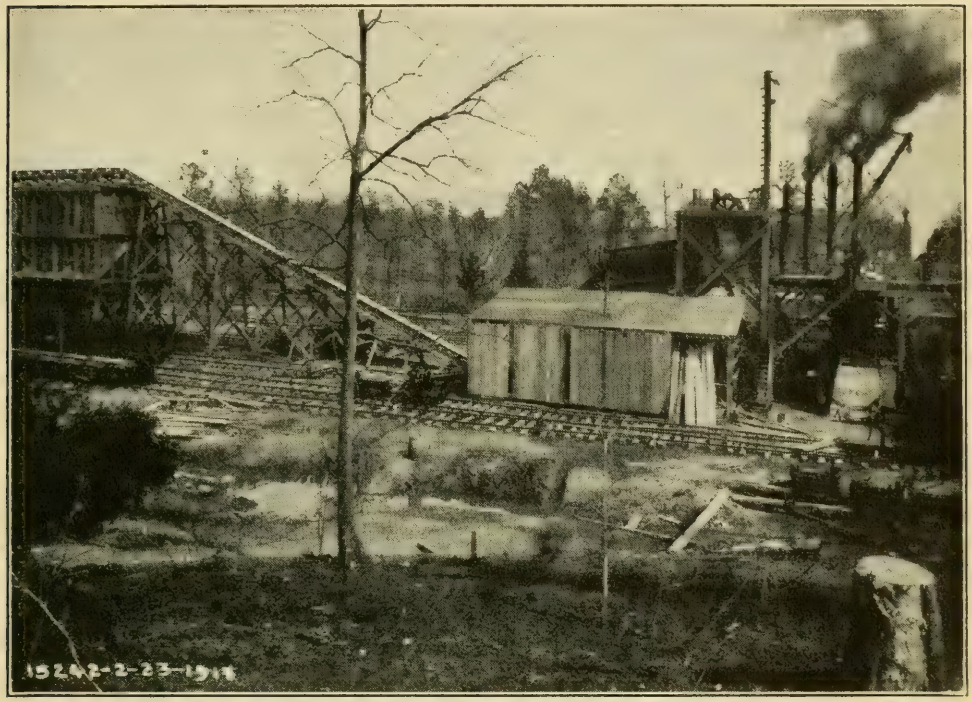


FIG. 21.—CRUSHER PLANT AT ZION QUARRY, SHOWING ALSO PORTION OF STORAGE BIN.

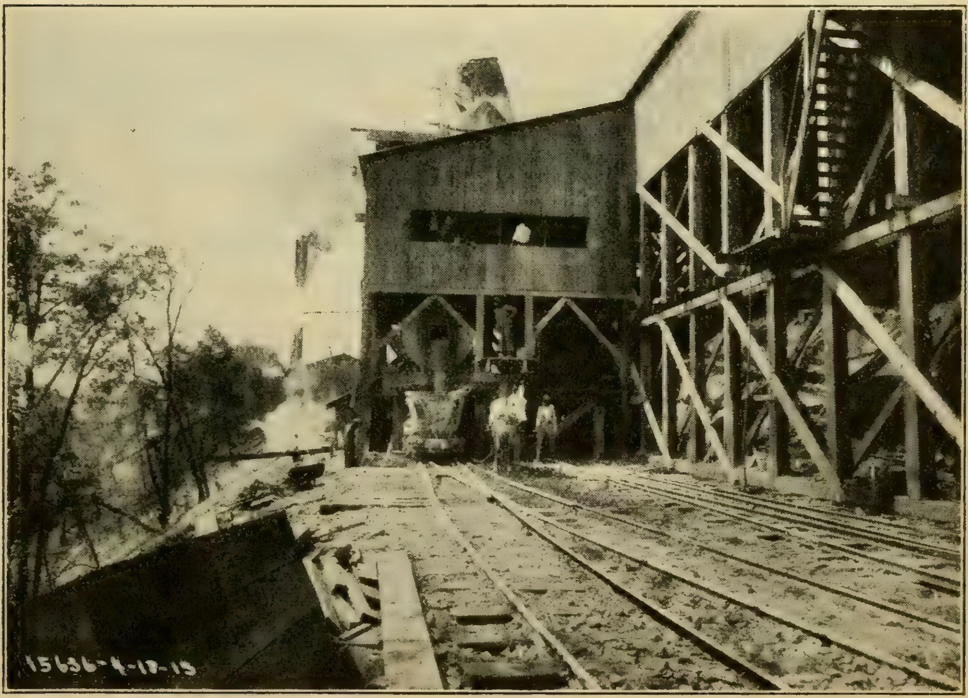
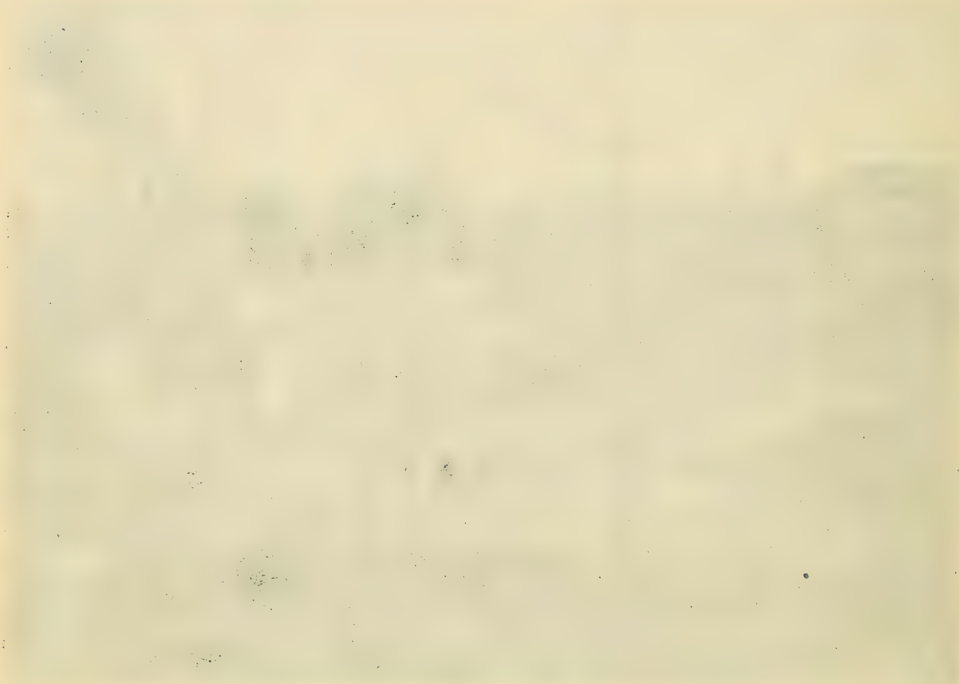


FIG. 22.—CONCRETE-MIXER PLANT.



yd. from Jackson's Lake to Ocampo, making the total cost per cubic yard at Ocampo, 95 cents. This sand proved satisfactory, although certain sections of the pits contained a good deal of loam, and it was necessary for the Power Company to maintain an inspector there constantly to supervise the loading and subject the sand to certain quick field tests for percentages of foreign matter.

All gravel secured from outside sources was washed, with the exception of that brought from the Company's pit at Elmore, Ala., which contained about 45% of gravel and 55% of sand. This pit was purchased to help out at the height of the concreting, and served its purpose.

The following tabulation shows the quantities of materials secured from various sources:

Crushed stone from Ellison Quarry....	11 031	cu. yd.
Crushed stone from Zion Quarry.....	115 207	" "
Cyclopean stone from Zion and Ellison.	7 670	" "
Stone purchased from outside sources.	13 998	" "
Washed gravel from outside sources...	31 031	" "
Elmore gravel (60% gravel).........	6 009	" "
Grand total....................	184 937	cu. yd.
The total quantity of sand used from all sources was..................	57 809	cu. yd.

Cement.—Cement was purchased from local mills, one in Alabama about 40 miles east of Birmingham and the other in Tennessee close to the Alabama line. It was tested at the mills by representatives of the Power Company, under the specifications contained in *Circular* No. 33 of the U. S. Bureau of Standards.

The cement was delivered to Ocampo in carloads of approximately 170 bbl., at a cost of $1.30 per bbl. net. At the dam site a weather-proof storage shed with a capacity of 15 000 bbl. was built and kept full until the work was nearly completed, when the storage was drawn on to tide over periods of car shortage caused by the cotton movement.

All cement, as well as gravel and sand, and that stone which came

from outside quarries, was contracted for by the Company and delivered to the contractor at Ocampo.

Lumber.—Rights to approximately 4 000 000 ft. of timber were acquired in the vicinity of the dam, and contracts were let locally to three small mills to cut it. All these mills were within a mile of the dam, and the sawed lumber was teamed by the Company to the point needed. It is to be noted here that, whenever possible, saw-mills should be on the construction railroad, or should have railroad service.

Timber proved to be very cheap in this section, the price of all the lumber used, of all classes, being about $10 per 1 000 ft. b. m. at the dam.

All the lumber for coffer-dams, camp buildings, bins, shops, stores, forms, etc., was supplied by the above timber and mills. Approximately, 6 600 000 ft. of lumber for all purposes—railroad, dam, and quarries—were used.

Commissary Supplies.—Practically all commissary and other running supplies were secured in the Montgomery or Birmingham market, being bought directly by the contractor. Before the construction railroad was ready for service, all these supplies were brought to Clanton, Ala., 13.6 miles from the dam, by road, and were teamed to the dam by local teams at 20 cents per cwt. Later, when the railroad was in service, a regular merchandise car was run by the Louisville and Nashville Railroad Company from Birmingham to Ocampo and thence taken to the dam on the Clear Creek Railroad. An arrangement was made with the Louisville and Nashville Railroad Company to run a regular package car exclusively for the work at the dam, and during the height of the work, there were two, and sometimes three, cars of merchandise daily to take care of the large force at work.

Coal.—Coal was contracted for by the contractor and delivered in hopper-bottom cars, *via* the Louisville and Nashville Railroad, at Ocampo, from which place it was carried on the construction railroad to the points desired. At the quarry and dam the coal was dumped through trestles directly in front of the boilers where it was to be used. On the railroad it was shoveled by hand out of the cars for engine uses directly into the tenders or to buckets moved by air hoists.

Coal was bought from various mines in Alabama, at a price of $1.50 per ton, at the mine, and a freight rate of $1.00, making the price, f. o. b. Ocampo, $2.50 per ton.

General Scheme of Handling Work at the Dam Site.—The general scheme and outline laid down and followed for the prosecution of the work contemplated the building of three coffer-dams and the handling of the work as follows:

1st.—Coffer-dam No. 1, which took in the west abutment, power-house, wasteway section, and 404 ft. of the dam, was built and the work within it excavated and concreted. This coffer-dam was designed and built to care for a flow of 75 000 sec-ft. in the river without flooding, and accomplished its purpose. Sufficient openings were left through the portion of the dam constructed within Coffer-dam No. 1 to care for a flow of 20 000 sec-ft. without flooding Coffer No. 2. The work within Coffer No. 1, it was anticipated, would be accomplished during the high-water stages of the river and that in Coffer No. 2 during the low-water, summer season.

2d.—The building of Coffer No. 2, turning the river through the culverts left in the first portion of the dam.

3d.—The construction of the second portion of the dam and east abutment, leaving more openings through it, with their bottoms at a higher elevation, in order to be able to pass a large flood if the river rose rapidly in the fall before the power-house end of the work was finished, and also making it possible for the reservoir to be drawn down if the water did rise to a considerable height before the work was completed.

4th.—The construction of the tail-race coffer-dam at about the same time as Coffer No. 2, and the excavation of the tail-race while the second section of the dam was being concreted. The power-house concreting was to be prosecuted while the first section of the dam was being built, but it was anticipated that it would be far behind that portion of the work, and would come to completion about the same time as the second section. Consequently, a small concrete wall, from the end of the power-house next to the wasteway section up stream, and a coffer-dam down stream to the tail-race coffer, were built in order that when the water was turned through the first section of the dam, the power-house and tail-race work would still be protected.

Section B—Methods Adopted for Handling the Work.

Camps.

The very first problem which presented itself in the prosecution of the work was the building of camps, and the providing of necessary messes and quarters for the men and houses for their families, as it was realized that unless there was proper food and comfortable habitations, it would not be possible to hold the men. It was anticipated that a very large force would be necessary in order to complete the work quickly and on time, and that, with such a large force, the sanitary and medical part of the work would need strict attention, and would be no small problem in itself. It was also essential that no contagious or infectious disease be allowed to gain a foothold and sweep through the camps, seriously hampering the work and giving the job a bad name. All camp buildings were constructed of wood, and were fairly well ventilated and cared for.

Lock 12 Camp.—At the dam five distinct sections of the camp were built. The white camp contained the commissary, white mess hall, bunk-houses, foremen's houses and families, and the houses of the contractor's staff, ice plant, bakery, storehouse, etc., in fact, this was the main camp. The engineers' camp was by itself, near the contractor's main camp No. 1, and contained a bunk-house, family dwellings of the staff, a mess hall, office, and a special house for the accommodation of Company guests and visitors, known as the "Guest House." The negro quarters were about $\frac{1}{5}$ mile from the main camp, on one side of the railroad. About 400 yd. from the negro quarters was a fourth camp, for foreigners. The Swedish carpenters lived in a camp by themselves. The camps for the negroes and foreigners were combined into one large negro camp after the foreign laborers at the dam were discharged. Fig. 23 is a general map of the Lock 12 yards, railroad connections, etc.

About 1 000 persons could be accommodated at the main Lock 12 Camp, and at the height of the work it contained about that number. The topography at Lock 12 prevented any general scheme of laying out the buildings; they were placed on the tops of the ridges, with the drainage away from them toward the river and creeks. Where possible, a regular arrangement should be adhered to, and is also recommended from a sanitary standpoint. Some of the bunk-houses at this camp were too large, and did not afford proper ventilation at

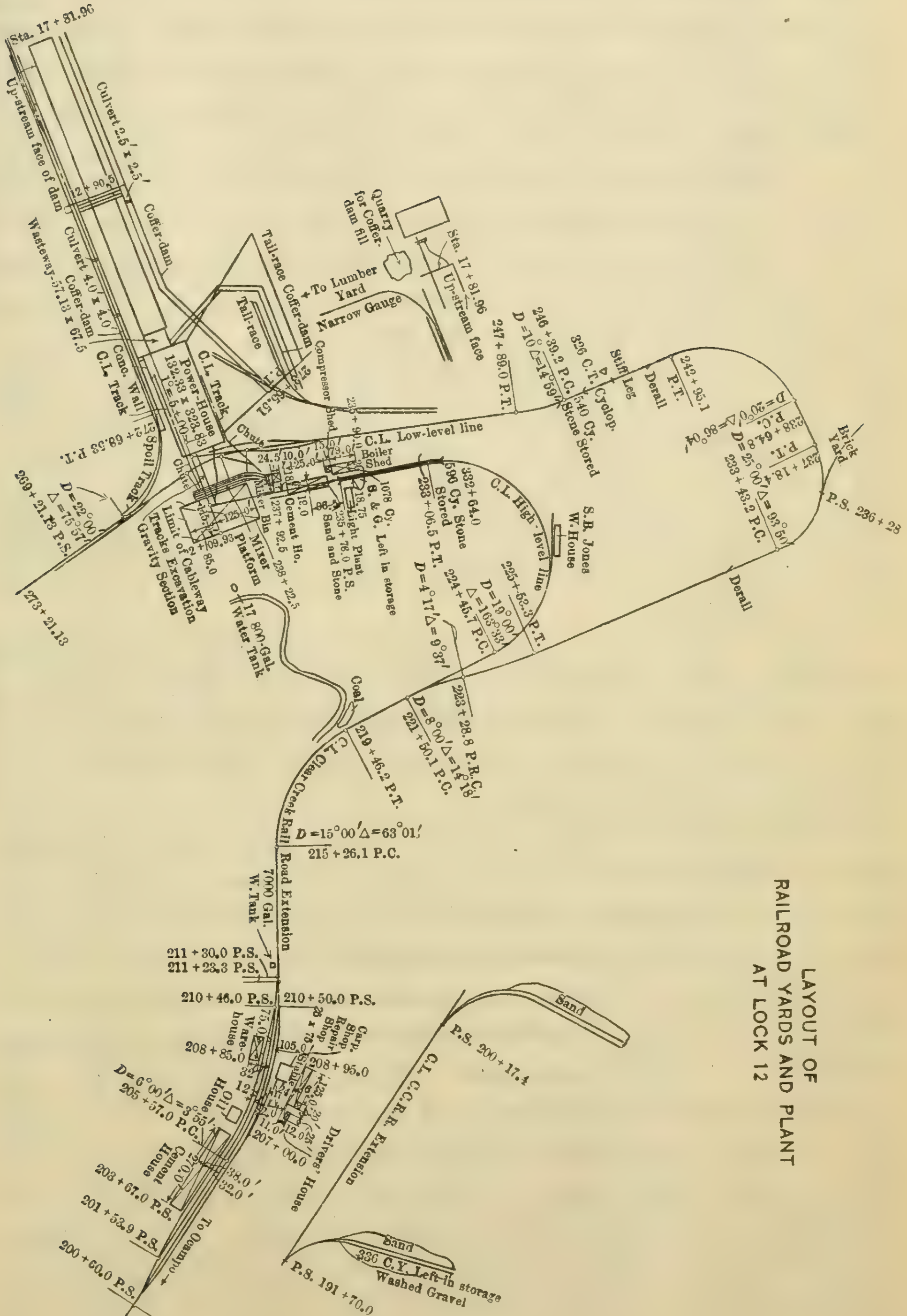

LAYOUT OF
RAILROAD YARDS AND PLANT
AT LOCK 12
Sta. 17 + 81.96
Up-stream face of dam
Culvert 2.5' x 2.5'
Coffer-dam
Wasteway-57.13 x 67.5
Culvert 4.0' x 4.0'
Conc. Wall
C.L. Track
Power-House
C.L. Track
Tail-race
Tail-race Coffer-dam
To Lumber Yard
Narrow Gauge
Quarry for Coffer-dam fill
Up-stream face
Compressor Shed
Boiler Shed
C.L. Low-level line
247 + 89.0 P.T.
Stiff Leg
Derail
P.T.
242 + 95.1
Brick Yard
P.S. 236 + 28
Spoil Track
Cement Ho.
Mixer Platform
Light Plant
S. & G. Left in storage
Stone Stored
C.L. High-level line
S.R. Jones W.House
Limit of Cableway
Tracks Excavation
Gravity Section
273 + 21.13
17 800-Gal. Water Tank
Coal
Derail
C.L. Clear Creek Rail Road Extension
219 + 46.2 P.T.
221 + 50.1 P.C.
223 + 28.8 P.R.C.
D = 15° 00' Δ = 63° 01'
215 + 26.1 P.C.
7000 Gal. W.Tank
211 + 30.0 P.S.
211 + 23.3 P.S.
210 + 46.0 P.S.
210 + 50.0 P.S.
Ware-house
Carp. Shop
Repair Shop
208 + 85.0
208 + 95.0
Oil House
205 + 57.0 P.C.
Cement House
Drivers' House
207 + 00.0
203 + 67.0 P.S.
201 + 53.9 P.S.
200 + 60.0 P.S.
To Quarry
P.S. 200 + 17.4
C.L. C.C.R.R. Extension
Sand
336 C.Y. Left in storage
Washed Gravel
P.S. 191 + 70.0

FIG. 23.

night. A house of proper size would be one large enough to accommodate twenty men, well ventilated, and, in the skilled and white quarters, there should be a shower bath in each house. In the negro camp, a large mess hall and several houses were provided. This, however, is not the best system to follow in handling the negro. A large number of small houses, with no general mess hall, the writers believe, would be better, and, later, this scheme was used, and is being used by other contractors on similar work in this section of the country to good advantage. To hold the negro laborer on the work it is essential that he have his negro women with him, as he will not stay in the camp unless the women are there. Consequently, provision has to be made to take care of them. Another feature of holding the negro is that he be allowed to shoot "craps" and play "skin", his favorite pastime and card gambling game, and it is useless to try to stop him. The local county authorities realized this here, and arrangements were made whereby the negroes did all their gambling and card-playing in the confines of their own quarters, where they were not molested.

In the Italian and foreign camps, no mess halls were provided, as laborers of this class prefer to board themselves. However, a bakery had to be established to provide the kind of bread especially preferred by them, and from 500 to 1 000 loaves per day were baked and distributed to the various camps.

In general, the buildings, as used, proved serviceable and efficient, but one error was committed, in a part of the camp, in placing them too close to one another. A fire in the negro quarters, which should have destroyed only one building, destroyed six, due to this cause.

In addition to the living and eating quarters described, a schoolhouse was erected for the white children, and used also for religious gatherings. A church was also built in the negro quarters. No recreation rooms were provided, and the writers consider this a mistake. A large, airy, reading and writing-room should have been provided for the white camp. It should have been in charge of a competent man, and operated at the Company's expense. It is the opinion of the writers that, with a place of this nature provided for the men, a certain amount of drinking and gambling can be eliminated, and, furthermore, the superintendent can put his hands on some of the men quickly in times of emergency.

A small 1½-ton ice plant was installed at the rear of the white kitchen at Lock 12, and, adjoining both the plant and kitchen, there was a large cooling room, where all meat for the camp was kept fresh. This plant did not prove quite sufficient for the Lock 12 camp in the hottest weather, and the supply for the other camps had to be contracted for from outside sources and hauled in daily by team. The plant proved satisfactory except as noted.

Water for the camp at the dam was secured from two sources. That used for drinking purposes came from drilled wells, 6 in. in diameter, and more than 130 ft. deep, one in the negro camp and one in the white camp. Water was circulated through the white camps from this well by a gasoline pumping outfit. For other purposes the water was pumped from Yellow Leaf Creek, ½ mile distant, by a steam pump, to a 20 000-gal. tank set on a tower in the highest part of the camp, from which mains distributed it throughout all the camps for fire, boiler, and locomotive purposes, baths, washing, etc. This water was not used for drinking on account of danger from typhoid infection.

Quarry Camps.—Houses of the same general types as those at the dam were used at the quarries, except that, in the Italian quarters, little kitchens were provided in which these people could do their own cooking. In the negro section of this camp there were tents, with floors and wooden sides half way up and permanent wooden frames. A commissary was run at Zion Quarry, which took care of both Ellison and Zion Quarries, and provided for the general mess hall and the individual messes. Bread was secured from the Lock 12 bakery, and ice, to keep the meat, etc., fresh, was hauled by team from Clanton. Ellison Quarry, while operating, had a small commissary run as a subsidiary to that of Zion Quarry. Water was provided at the quarries by wells, from 100 to 150 ft. deep, drilled with a regular well-drilling rig.

Lighting of Camps.—Steam-driven lighting plants were provided at the dam and at Zion Quarry, and furnished lights to all houses, shops, storehouses, etc., and the general camp and job lighting. Dwellings were supplied with light, at an equitable rate, if so desired. Flaming arc lamps were used on the work proper and for general lighting throughout the camps. Later, when the transmis-

sion lines were connected up to Gadsden, the steam plant was shut down at Lock 12, and current was secured from this source.

Hospital, Medical Service, Sanitation.—All medical and hospital service was handled under a contract with a local physician who lived at the dam. Under the contract he was to render services at all the camps, provide all drugs, medicines, a hospital, nurses, etc., and receive in return 90% of the collections from the men, which amounted to $1 per month per man.

The doctor also acted as sanitary officer, but merely reported what he saw and had no authority to enforce his rulings, which was not a desirable feature. Sanitary precautions were taken by providing the pail system of closets and making the boxes of the closets fly-proof. The refuse matter was gathered nightly by a sanitary wagon, hauled a considerable distance from camp, and buried. All closets were kept well sprinkled with lime at all times. All kitchens and dining-rooms were screened, as well as practically all the family dwelling-houses.

The general health of all the camps was remarkably good. No contagious or infectious diseases originated at any of them. At the quarry camp several cases of typhoid fever developed among the Italians, and was traced to some bad meat which they had secured from an outside source. Only about six or eight cases came from this source, one of them being an inspector on the Resident Engineer's staff.

All accidents were treated immediately by the resident physicians, at the camps in which they occurred, and serious cases were brought to Lock 12 to the general hospital, where nurses were maintained.

Policing the Camps.—A regular force of deputies was maintained by the Power Company, regularly commissioned under the Sheriff of Chilton County, there being two deputies at Lock 12, one at the quarry, and one on the railroad. The most troublesome features arose from bootleggers bringing whiskey into the camps.

Camp Bosses.—Camp bosses looked after the cleaning and care of the camps; and their assistants, known as shack rousters, looked after the cleaning of the bunk-houses, etc. In the negro camp, the camp boss was a responsible white man who thoroughly understood the negroes and saw that they were not allowed to lie around

the camp, unless actually ill. He kept track of the various families occupying the small houses, looked after the checking of the dining-room, kept his eye on bad negroes, and, in general, maintained order. He also assigned quarters to new men coming in and looked after them, seeing that they got out to work properly.

Layout of Contractor's Plant and Details of Various Main Parts of the Plant.

Railroad Yards and Tracks.—At Lock 12 extensive yards and tracks for the storage and rapid handling of materials were provided; and a line was built down into the river bottom, below the dam, whereby trains and engines could get down on the cofferdam; this was known as the "Low-Level Line." There were two storage yards for sand and gravel where loading and unloading operations in no way interfered with regular train and switching movements. A long side track served the main cement warehouse, oil house, and general store-house, so that cars were unloaded directly into them. The shops were also served with another siding, but the arrangement used was a poor one, and detailed criticism of this feature of the layout is given elsewhere. The main line, known after it left the yards on its way to the dam as the "High-Level Line", served the mixer, cement warehouse, concrete material bins, and compressor plant, materials being unloaded directly to these various places from the cars. Fig. 23 is a map showing this detailed track layout.

Cableways.—Two traveling cableways spanned the river parallel to and immediately over the face of the dam. Together, the two cableways had a range of about 150 ft. up and down stream, that is, one could be placed over any portion of a strip 150 ft. wide at right angles to the line of the dam. The towers of these cableways were 95 ft. above the rail, their track elevation being about 440. The length of span, from center to center of towers, was 1 647 ft. The towers traveled on standard-gauge tracks and on standard M.C.B. car wheels and axles. The main cables were $2\frac{1}{4}$ in. in diameter and of patent, lock-laid, flat, steel strands on the outer covering. The cableways were also equipped with one button line, having eight buttons on it, one endless line, and one hoist line. The button and hoist lines were $\frac{3}{4}$-in., $\frac{6}{25}$-strand, Hercules, wire ropes.

The endless line was a $\frac{3}{4}$-in., $\frac{6}{25}$-strand, flat, wire rope. The button line was 1 700 ft. long, the hoist line, 2 100 ft., and the endless line, 2 600 ft. long. The cableways were operated by $12\frac{1}{4}$ by 15-in. engines, driven by steam, supplied from 80-h.p., locomotive-type boilers, all mounted on the trucks of the towers. The cableways had a rated capacity of 10 tons, and were operated under a sag of 80 ft. with a load of 6 tons.

In the early stages of the operation of the cableways, a good deal of trouble was experienced with carriers. The original carriers seemed to be too heavy, frequently breaking each other when they struck. Lighter carriers were made, and much of the trouble was eliminated.

Trouble was also caused by the breaking of two strands in No. 2 cable. The first breaks were welded by the oxy-acetylene process. The weld did not prove satisfactory, and they broke again; this cable was used for the remainder of the job with these strands cut out.

The buttons used on the button lines were of an obsolete type, and were not satisfactory; when one of them broke or slipped along the cable, it was necessary, in order to put on a new one, to lower the cable, cut out a section, and replace it, all of which was expensive and caused much delay. There is now manufactured a cast-steel button, made in two parts, which does away with the necessity for lowering the cable. A button of this type can be put on in an hour, and is a great improvement over the old one.

Extra hoist, button, and endless lines were kept in readiness at the head-towers, so that, in case of a breakdown at any time, they could be quickly and rapidly installed. The cableways handled practically 60% of the concrete, and were run both day and night. They also handled practically all the excavation in the early stages of the work. They were used for moving and erecting all derricks, etc., handling form lumber, iron, and materials of a like character, and were found to be exceedingly useful for the latter purpose.

Derricks.—All derricks used at the dam for excavating and placing concrete were of 5-ton capacity, of the wooden, traveling, stiff-leg type, with 37 to 40-ft. masts and 55 to 65-ft. booms. These derricks traveled on the up-stream and down-stream coffer-dams, parallel to the line of the dam, and on the bottom in the tail-race

and draft-tube excavation of the power-house. Later, when the concrete reached Elevation 380.0 in the power-house, three of these derricks were set up on high bents and were stationary.

All the derricks were equipped with $\frac{5}{8}$-in., $\frac{6}{19}$-strand, Hercules, steel-wire rope hoist and boom lines, and with 14-in. derrick irons and 12-ft. bull-wheels. Double blocks were used on the booms and single blocks on the hoist lines, and were operated by 7 by 10-in. engines, with swinging gears, driven by air. The traveling feature of these derricks seems to be an excellent one, as their increased radius of action is most advantageous. In this case the back trucks of these derricks traveled on the top of the coffer-dam, and the front trucks on a trestle resting on the bottom inside. Three of them were able to cover a length of 475 ft. of the dam, on the up-stream face. These derricks handled excavation in skips into cars run out on the up-stream coffer, and took concrete from the cableways, placing it in the forms, the cableways depositing a loaded bucket in a certain place and removing the empty bucket, the derrick then taking the bucket and dumping it in any portion of the form desired. They also took concrete from flat cars run out on the up-stream coffer-dam.

Mixer Plant.—The mixer plant was about 200 ft. south of the cableways, and discharged concrete to cars standing at Elevation 425. The plant consisted of two Model 64, 84-cu. ft. mixers, owned by the Power Company, and driven by an 18 by 24-in., horizontal, single-cylinder engine, rented from the contractor. This engine was run by steam supplied from the compressor plant, about 100 ft. away, as shown in Fig. 22. These mixers had a capacity of 20 batches per hour, their combined capacity being 3 200 cu. yd. for two 10-hour shifts. The highest run made was 2 220 cu. yd. Materials for concrete came in in hopper-bottom cars, and were dumped through a trestle into a bin having a capacity of 800 cu. yd. They were removed from this bin by a belt conveyor to a bucket elevator, and deposited in smaller bins over each mixer, from which they were drawn to the measuring hoppers. The belt conveyor and bucket elevator were driven by a 10 by 16-in., horizontal, single-cylinder, engine, also run by steam supplied from the compressor plant. The drive on both the mixers and elevators was with belts, to line shafts, and friction clutches. One of the drums of the mixers wore through,

comparatively quickly, due to the exceedingly tough and hard quality of the stone. This drum mixed approximately 70 000 cu. yd. of concrete. These drums were of ¼-in. metal, and the writers believe this to be somewhat too thin for this size. They suggest that a ⅜-in. shell would be preferable. Four sets of blades and one drum were worn out on one mixer, and three and one-half sets of blades on the other, in the course of the job.

Two elevators were installed during the progress of the work. The first was a No. 6, 18-in. bucket, stone elevator, 67-ft. centers, on the main pulleys. It did not have sufficient capacity to supply 1 000 cu. yd. per 10-hour day to the mixers, as required, and was taken out in August and replaced with a No. 8, 30-in. bucket elevator. A 32-in., 6-ply belt, 147 ft. long, was used, and had ninety-eight 30 by 17 by 10-in. steel buckets. After the installation of the larger elevator, there was no more trouble.

Water.—Water was supplied to the mixers through a 2-in. line from the main storage tank at the dam. Two small iron tanks with the necessary control valves, gauge glasses, etc., were equipped and set up overhead in the mixer-room, and all the water was drawn directly from these to the mixers.

Empty Sacks.—As soon as the cement was emptied into the hoppers, two laborers assigned to the duty gathered the sacks, shook them off to one side, packed them in bundles of 50, and stored them in the mixer cement house until from 20 000 to 30 000 sacks had accumulated, when a carload was shipped to the mills. The sweepings were sacked and used with the other cement.

Compressor Plant.—Practically all derricks, hoists, small pumps, air drills, small wood-boring tools, etc., were operated by air at a pressure of 85 lb., furnished by a central compressor plant which was near the mixing plant. The boilers of this plant also furnished steam for driving the mixing plant.

Two 24 by 30-in. straight-line air compressors, each with a capacity of 1 225 cu. ft. per min., furnished all the air. These machines were leased with other equipment from the contractor. The compressor and the mixer plant were supplied with steam by four 125-h.p. horizontal, locomotive-type boilers, leased from the contractor.

The compressors delivered air at 85 lb. pressure through a 6-in. main to a storage tank, 4 ft. 9 in. in diameter, 12 ft. long, and of

$\frac{5}{8}$-in. metal, located outside the building. A system of pipes was arranged to spray water constantly over this tank to keep it cool, and a special grade of oil was used in the compressors on account of the likelihood of explosion in the storage tank. The air was distributed to the principal points of use through a 6-in. main.

The construction of the compressor plant was commenced on November 16th, 1912, and the first compressor was turned over on January 14th, 1913. The plant was entirely completed and ready for business on February 1st, 1913.

Machine and Carpenter Shop.—The machine, carpenter, and blacksmith shops were all in one long building in the yards near the main storehouse. One line shaft, 84 ft. long, ran through the length of the building, and all machine tools were driven from it. The engine driving this line shaft was an 11 by 14-in., horizontal, single-cylinder engine owned by the Alabama Power Company. It was supplied with steam by a 30-h.p., vertical boiler, leased from the contractor. The engine and boiler-room was in the center and at the rear of the shops.

The machine shop contained the following tools:

One 30-in. radial drill press,
One 24-in. shaper,
One 18-in. by 10-ft. lathe,
One $1\frac{1}{2}$-in. bolt threading machine,
One No. 94 Forbes, $2\frac{1}{2}$ to 6-in., pipe cutting and threading machine,
One 16-in. by 5-ft. lathe,
One 30-in. diameter by 5-in. face power grindstone, and
One No. 4 Vulcan emery grinder.

The blacksmith shop contained the following:

One No. 2 Champion blower for forges,
Three blacksmith forges, home made,
Three anvils, from 250 to 300 lb.

The carpenter shop contained:

One No. 8 variety rip-saw and joiner,
One band saw,
One No. 402 bench grinder,

One No. 8 planer and matcher. This planer was very useful, and surfaced all form lumber; if the saw-mill had been operated by the Company, however, the planer should have been there.

The writers consider that the foregoing equipment served its purpose well, but there should have been added to the machine-shop a power hack-saw, a set of shears large enough to cut rounds up to 1 in. in diameter and flats up to 1 by 3 in., and a small steam-power hammer, on a job where there are a number of locomotives and cars to be taken care of.

The shops, as located, did not have sufficient floor and working space, and should have been separated from each other. Both should have been closer to the dam, and served by independent spurs from the railroad, so that cars of lumber could have been delivered to the platforms and yards direct. The laying-out floor of the carpenter shop should have been large enough to build one complete scroll case form on it, which would have taken a platform 75 ft. square. Also, there should have been a second laying-out floor, at least 50 by 100 ft., for ribs and other work of that kind.

The machine shop, if possible, should be as close to the work as convenient for all purposes, and should have at least two tracks for its own exclusive use: one for bringing machinery, etc., to be repaired at the shop; the other to serve as a general rip yard track. Where a railroad is operated and engines are taken care of, a pit for dropping wheels should be provided on one of these tracks.

Locomotive Cranes.—Two 10-ton locomotive cranes were leased from the contractor, and used in all features of the work. They did the work well, but the type used had only four wheels, which was hard on the track when traveling around. Two 1½-yd. clam-shell buckets were furnished with these cranes, and were used for unloading, storing, and reloading from storage all sand and gravel, and for coaling engines, etc. They were also used to assist in the disposal of excavation from the dam and tail-race, handling loaded skips. They were also used in unloading and erecting the contractor's plant and in reloading it when the work was completed. They were found exceedingly useful all through the work, but were not quite big enough for all purposes. After the work was started a larger crane was purchased, having a 50-ft., fixed boom and a 30-ft., trussed extension. This crane had a capacity of from 20 to 30 tons at 12 ft. radius, and

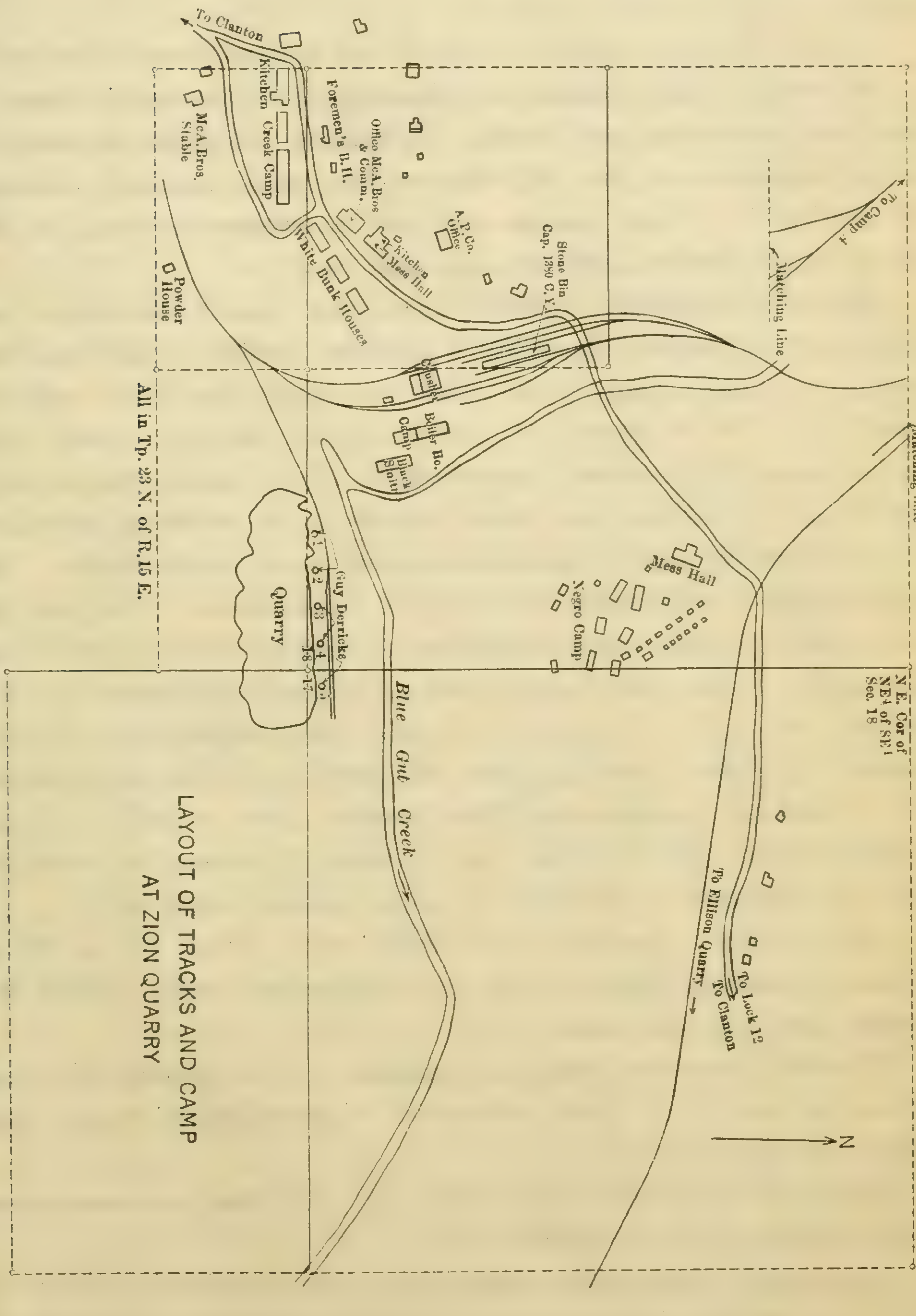

To Clanton
Kitchen
Creek Camp
McA. Bros. Stable
Foremen's B. H.
Office McA. Bros & Comm.
A. P. Co. Office
Stone Bin Cap. 1390 C. Y.
Kitchen
Mess Hall
White Bunk Houses
Powder House
Crusher
Boiler Ho.
Camp
Black Smith
Matching Line
To Camp 4
Matching Line
Mess Hall
Negro Camp
Quarry
Guy Derricks
All in Tp. 23 N. of R. 15 E.
Blue Gut Creek
N. E. Cor of NE¼ of SE¼ Sec. 18
To Ellison Quarry
To Lock 12
To Clanton
N
LAYOUT OF TRACKS AND CAMP
AT ZION QUARRY

FIG. 24.

was used at first for stripping at the quarry and loading cyclopean stone. Later, it was used in erecting the steel framework of the power-house and handling the heavy castings and machinery going into it.

Quarry Plant and Layout.—Fig. 24 is a general map of the layout at the quarry. The following is a description of the various pieces of plant and machinery used there: In general, the plant consisted of a crushing outfit of two McCully crushers, with necessary elevators for storing in a large storage bin, and a compressor plant for the operation of derricks, drills, pumps, etc.

Crushers.—Material brought from the quarry in skips was dumped by a derrick into a No. 9 gyratory crusher, leased from the contractor. After passing the No. 9 crusher, the stone was elevated by a No. 9, geared-head stone elevator having a 36-in. belt, traveling on 32-ft. centers, approximately 75 ft. long, equipped with fifty-two 37 by 18 by 11-in. steel buckets, to a revolving screen, 60 in. in diameter and 12 ft. long, mounted on a wooden frame. The perforations in this screen were 3½ in. in diameter. The belt, elevator, and screen part of the plant was owned by the Alabama Power Company. Stone which would not go through the screen was discharged into a No. 7½ gyratory crusher, also rented from the contractor. The final crushings were delivered to a 30-in., geared-head, belt conveyor, traveling on 270-ft. centers, over a 30-in. self-propelled tripper. The belt was 552 ft. long. The first belt wore out and was discarded in October for one of canvas, stitched.

The crushers, elevators, and screens were driven by an 18 by 27-in., horizontal, single-cylinder, non-condensing engine, which was rented from the contractor.

The belt conveyor from the crushers, to the storage bins was driven by an 8 by 10-in., horizontal, single-cylinder engine, purchased by the company.

Steam for both these engines was supplied from the compressor-plant boilers, a short distance away.

Compressor Plant.—The compressor plant consisted of four 18 by 24-in., straight-line compressors, having a capacity of 670 cu. ft. per min., and were rented from the contractor. This plant could operate a maximum of nine drills and six derricks. The plant was

not quite large enough for the needs on this work, and one more boiler and compressor would have been more satisfactory.

The compressors and other engines were supplied with steam by four 100-h.p., and three 75-h.p. horizontal, locomotive-type boilers, all leased from the contractor.

A coal trestle, 12 ft. high, was built immediately in front of the boiler-room, and hopper-bottom cars of coal were shunted to the top of it and dumped through. The coal was thus unloaded with a minimum expense at the point of use.

Derricks.—For handling stone out of the quarry proper there were six latticed-frame, steel-guy derricks, three of which had 75-ft. masts and 70-ft. booms, and were owned by the Power Company; and three had 65-ft. masts and 60-ft. booms. Each had a capacity of 10 tons.

These derricks proved to be somewhat weak for the work, and the bottom third of the mast had to be reinforced with four 30-ft., 60-lb., steel rails. These derricks were also very difficult to repair, for when a boom fell, or the derrick itself fell, it was generally badly bent. Timber derricks would have been better in this section, where a 70-ft., long-leaf, yellow pine stock, with a 12-in. top could be obtained for $25.

There was also one timber guy derrick, with a 73-ft. pine mast and a 68-ft. pine boom, using 14-in. derrick irons, the timber for it being cut from the Company's land. This derrick was placed at the crushers, and handled skips of stone from flat cars to the crushers.

All the engines on these derricks were 7 by 12-in., 24-h.p., double-cylinder, double-drum hoists, and fitted with size 3½ Dake swinging engines. This rig proved satisfactory, and was considered a good one for the work.

Miscellaneous.—For washing the crusher muck before it went to the crushers, and cyclopean stone, a No. 11, 6 by 10½ by 18-in. pump, steam-driven, was set up in the boiler-room. It had a capacity of 450 gal. per min., and furnished water, at 60-lb. pressure, through three 2-in. hose lines with 1-in. nozzles. The men stood on a washing platform and directed the water on the skip loads of stone just before they reached the crushers.

For pumping the quarry pit an 8 by 5 by 13-in. pump, having a capacity of 100 gal. per min., was found amply sufficient.

The heavy drilling in the quarry was done with No. 14, gasoline. portable, blast-hole, well drills. These were driven by a single-cylinder, gasoline engine, and drilled a $5\frac{5}{8}$-in. hole, the whole outfit being mounted on the same frame, on trucks.

The lighter drilling was done with Ingersoll-Rand, E 24, rock drills, equipped with No. 44 air heads, No. 42 chucks, and No. 27 tripods, with weights.

A blacksmith shop with two forges was maintained at the quarry for the running work there, and also a No. 3 Leyner drill sharpener, which took care of the sharpening of all drill steel. It was operated by air, and was a great money and time saver. Where a great deal of drill work is done, and there is a large quantity of steels to keep sharpened, a tool of this kind should be used.

As a whole, the quarry plant was satisfactory and the layout good. The plant had a capacity of 2 000 cu. yd. of crushed stone per day, but the largest day's run was a little more than 1 000 cu. yd.

Storage and Care of Materials.

Two yards were provided for the storage of sand and gravel. and were served by spurs leading off the main line of the railroad west of the yards at Lock 12, as shown on Fig. 23. Each spur branched into a double track after leaving the main line, so that a locomotive crane operating on one track could unload a string of cars on the other without having them switched. Structural steel was also unloaded and stored in one of these yards. In addition, 1 600 cu. yd. of sand were dumped through a high trestle next to the compressor plant and about 300 ft. from the mixer bins. Run-of-pit gravel (containing about 55% sand) from Elmore was stored in the same piles with the sand. Washed gravel was stored in separate piles. At the quarry a storage of 8 000 cu. yd. of crushed stone was accumulated during the latter part of the job, when the concreting slacked up at the dam. This stone was stored beside the main line of the quarry branch, being dumped from 7-yd. cars over the side of a 10-ft. embankment. Storing this material at that place permitted the quarry to shut down, thus getting rid of a large overhead expense some time before the concreting at the

dam was finished. This material was afterward loaded by a locomotive crane operating a ½-yd. clam-shell bucket. A maximum quantity of 8 400 cu. yd. of sand was in storage on December 14th, 1913, and approximately 2 000 cu. yd. of Elmore gravel, and 2 800 cu. yd. of washed gravel. These quantities of materials were put into storage primarily in order to avoid hauling over the railroad any more material than was absolutely necessary after January 1st, 1914, the rainy season generally starting about then and giving a great deal of trouble from soft track, derailments, etc.

Cement Storage.—Cement was stored in two warehouses, the main one being in the Lock 12 yard, and having a capacity of 15 000 bbl. This building was absolutely damp-proof, and the cement stored there was well preserved. The other warehouse, accommodating about 2 000 bbl., was built at the mixers. This storage fluctuated considerably, as it was used simply to insure a supply of cement at the mixers at all times. Most of the cement used at the mixers was unloaded there directly from cars and was not kept in storage.

Sand.—So much loam and foreign matter was contained in certain sections of the sand pit that it became necessary to place an inspector there. He sampled the sand from fixed positions in the cars and made the ordinary rough test of mixing it with water in large test tubes and allowing the material to settle after being thoroughly shaken up. The coarse sand settling to the bottom was measured, and the percentage of clay or loam resting on its surface was determined. Cars containing more than an average of 10% of clay or loam, as determined by this method, were not accepted. This percentage was greater than is ordinarily allowed, but there were times when no cleaner sand could be obtained, and many carloads were rejected. The concrete obtained with the sand was very dense and of excellent quality. A very close watch was also kept on foreign crushed stone, as some of it was dirty; if it was bad, it was rejected, by an inspector at Ocampo, before being brought to the dam.

Watch was also kept and inspection made of incoming materials at the dam. The engineers in charge on the dam made regular trips during their respective shifts to the bins and the mixers, and kept close watch on the character of the materials running.

A good deal of trouble was caused by using washed sand from foreign pits by itself; being very coarse, it would not hold the cement

or water, and would not dump out of the buckets, thus causing considerable delay at times. The principal remedy was to dump cars of fine sand containing some loam alternately with the coarse sand, mixing it as much as possible in this manner. About 30% of this coarse sand was retained by a ⅛-in. screen.

A series of tests was run on the Elmore gravel, and also on the washed gravel, to determine its fineness; 50% of the washed gravel would pass a ½-in. screen, and 55% of the Elmore run-of-pit gravel would pass a ⅛-in. screen.

The quantities of the various materials were determined both by measurement and by weight. All materials coming through Ocampo were weighed on a standard track scale installed by the Power Company. One hundred cars of material of each class were weighed and also measured, and the weight per cubic yard was thus determined. This factor was then used on all subsequent receipts. All quarry-crushed stone was measured in cars at the quarry by the inspector. All cyclopean stone was measured on cars by the yard clerk at Lock 12.

Section C—The Prosecution of the Work.

Railroad.—Owing to the site of the dam being 12 miles from the nearest railroad, one of the first problems to be solved was that of building a construction railroad. Time was very limited in which to construct this railroad to the site of the dam, as it was August when it was decided to start the work at Lock 12, and a railroad, under the contract, was to be completed to the dam by November 20th, 1912. As a matter of interest, plant was delivered there on November 26th. This short time allowance was practically the governing feature in the selection of the route adopted.

An old abandoned lumber road, known as the Clear Creek Railroad, extended from Ocampo, a point on the Louisville and Nashville Railroad 39 miles from Birmingham (Fig. 3), toward the dam, a distance of 17 miles. By extending this road 5 miles, the site of the dam was reached, making a total length of 22 miles. Surveys and investigations indicated that this would be the best route by which to get a railroad to the dam in at least 60 days. It was necessary to rebuild about 45 trestles on the old road and to re-tie it. The rails were still on the original ties, never having been taken up.

An inspection of Fig. 3 at once brings up the question: Why was the line not brought down from Ellison Quarry along Yellow Leaf Creek to the dam site, thus reducing the haul from the quarries materially? The answer is that, at the time Zion and Ellison Quarries were selected, the main line had been extended nearly to the dam, and about 80% of the work on the extension was done. If the quarries had not been at Zion and Ellison, the natural route for the line would have been as built.

In August the Power Company started a force of men from Ocampo to rebuild the old line, and a large grading outfit started to grade at the end of this line toward the dam. The work was completed, and steel was laid to the dam by November 26th, when carloads of plant were moved in. The roadbed, however, was still in a very bad condition, and 3 months more were spent with heavy forces re-tieing, surfacing, and opening up the old drainage channels which had become clogged. A steam shovel was put in, and the last 12 miles of the line on the dam end was ballasted, with shale taken out by the shovel and with crushed stone from the quarry. The first 10 miles of the road ran through a very rough hilly country, and the line was exceedingly crooked and full of heavy grades, though the roadbed was good and needed no ballast. On this section the maximum curve was 35° on a $2\frac{1}{2}\%$ grade, the maximum grade being 4 per cent. On account of these physical conditions, Shay engines were used entirely on this division, five in all being purchased or rented.

The operation of the railroad was quite a problem in itself. In the height of the season more than 250 men were employed on it alone, and on some days more than 100 trains of all classes were operated. The road was divided into two distinct divisions, the first 10 miles being the Shay Division, using the Shay engines only, and the last 12 miles, including the Quarry Branch, operated by rod engines. During the period of greatest activity, outside of switching service, five Shay engines and six rod engines were used. Operating out of Ocampo, a 40-ton Shay engine could haul only 2 cars of sand or 3 cars of cement to a train. On the rod-engine division, 5- and 6-car trains were hauled with 60-ton engines.

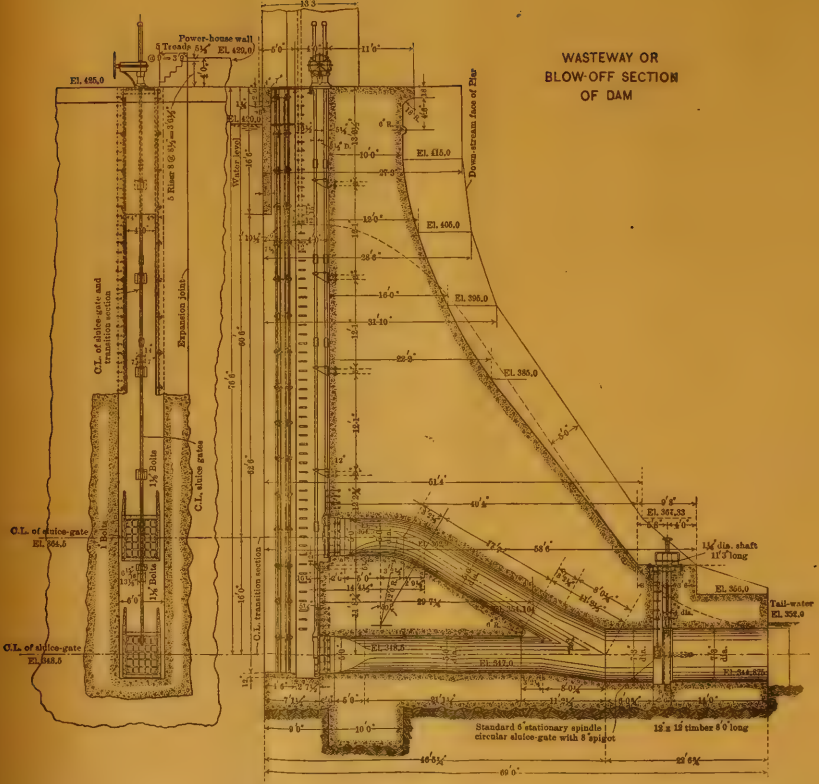
On the rod-engine division, 8 miles from the dam, a branch line ran down to the quarries, a distance of 2 or 3 miles, and the heaviest movement was between this junction and the dam. Railroad head-

quarters were established at this point. A regular camp, similar in character to all the others, was built, shops were put up to take care of the rod engines, and a commissary was maintained. The superintendent, master mechanic, car repairers, and all rod division trainmen, hostlers, shopmen, etc., lived there. All Shay division trainmen, machinists, etc., lived at Ocampo, where there was a smaller camp; a well-equipped shop had to be maintained at this point, also, to take care of the Shay engines.

The Shay engines gave a great deal of trouble, their maintenance costs were high, and this division of the railroad was the most troublesome to operate. The use of Shay engines here was compulsory, as no other type could work on the grades and curves of the first 10 miles of the line. In view of the experience gained here, their use should be avoided, if possible. If a new line is built, it should be laid out for the use of rod engines throughout, and for heavy traffic. In the original road at this place 58¼-lb. steel rails were used, but were too light. The writers would recommend nothing lighter than 75-lb. steel where traffic is handled directly in the original cars from a connecting railroad, because, in such cases, net loads of 50 and 60 tons will have to be taken care of in modern cars of 100 000 lb. capacity. The tracks should be well ballasted and the roadbed should be standard in every respect.

The time element entered largely into the selection of the railroad as used, making it almost compulsory. Had there been more time, it is quite probable that an entirely new and independent line would have been built from some near point on the Louisville and Nashville Railroad to the dam. The building of such a line, however, would have delayed the work at the dam from 60 to 90 days, and it was not considered good policy to do this, with the short time available in which to comply with Government requirements as to the completion of the dam.

Stream Control.—The first essential feature of the work that had to be studied and carefully worked out in detail was that of the control of the river. All indications and records of the behavior of the Coosa for 15 years previous indicated that this in itself would be no small part of the problem, and that its solution would have a most important bearing on the progress of the work. All the Government records for a period of 15 years previous to 1913 indicated a


WASTEWAY OR
BLOW-OFF SECTION
OF DAM
Power-house wall
El. 425.0
El. 420.0
El. 415.0
El. 405.0
El. 395.0
El. 385.0
El. 367.33
El. 356.0
Tail-water
El. 352.0
Down-stream face of Pier
C.L. of sluice-gate and transition section
Expansion joint
C.L. sluice gates
C.L. of sluice-gate
El. 364.5
C.L. of sluice-gate
El. 348.5
Water level
C.L. transition section
El. 318.5
El. 347.0
El. 344.875
1¼ dia. shaft
11'3" long
Standard 6" stationary spindle
circular sluice-gate with 8" spigot
12" x 12" timber 8'0" long
69'0"

which was approximately 4 ft. below the top of the large openings left in Section No. 1. These openings were left as a positive safeguard to the power-house end of the work. This portion of the work was much slower than that on the dam, and it was feared that it would hang over until January, which was a very doubtful season, and a month in which a great deal of water might be expected. It was feared that, with only the eleven 15-ft. openings, a heavy and protracted flood would back up the river and there might be a rise sufficiently high to flood out completely the power-house work. The ten 8 by 12-ft. openings would have prevented this and, at the same time, helped to draw down the river more rapidly, if it rose to a considerable height.

Two types of gates were used to close these openings, a detailed description of which is given elsewhere. Figs. 15 and 16 give an idea of how these openings were arranged and closed.

Coffer-dams.—The construction of Coffer-dam No. 1 was begun on September 11th, 1912. It was composed of cribs 20 ft. square, roughly divided into two compartments; an 8-ft. compartment on the outside for clay and loam, and a 12-ft. compartment on the inside for rock. The cribs were built on the shore, launched, towed to their places, and sunk. After getting about 300 ft. out from shore, it was found necessary, on account of the increasing current, to stretch a ¾-in. wire rope across the river, by which the cribs were trolleyed out and placed. Before building a crib, soundings were taken, and the bottom timbers of the crib to be placed at that point were framed to conform as closely as possible to the shape of the bottom of the river. The coffer-dam was built of separate cribs to about Elevation 353. The joints in the timbers from that elevation to the top were then broken in the center of the cribs so as to make it as strong and as near one unit as possible. Sheeting, consisting of two thicknesses of 2-in. plank with lapped joints, was then placed on the outside of the cribs, and a toe-fill of sand, gravel, and loam, with a slope of about 1 : 1, was placed around the complete dam on the outside. Rock for filling and weighting the cribs of Coffer No. 1 was secured from a slate quarry about 200 ft. up stream from the dam, on the west bank of the river. Owing to the extreme swiftness of the current, the east or river end of Coffer No. 1, running up stream and down

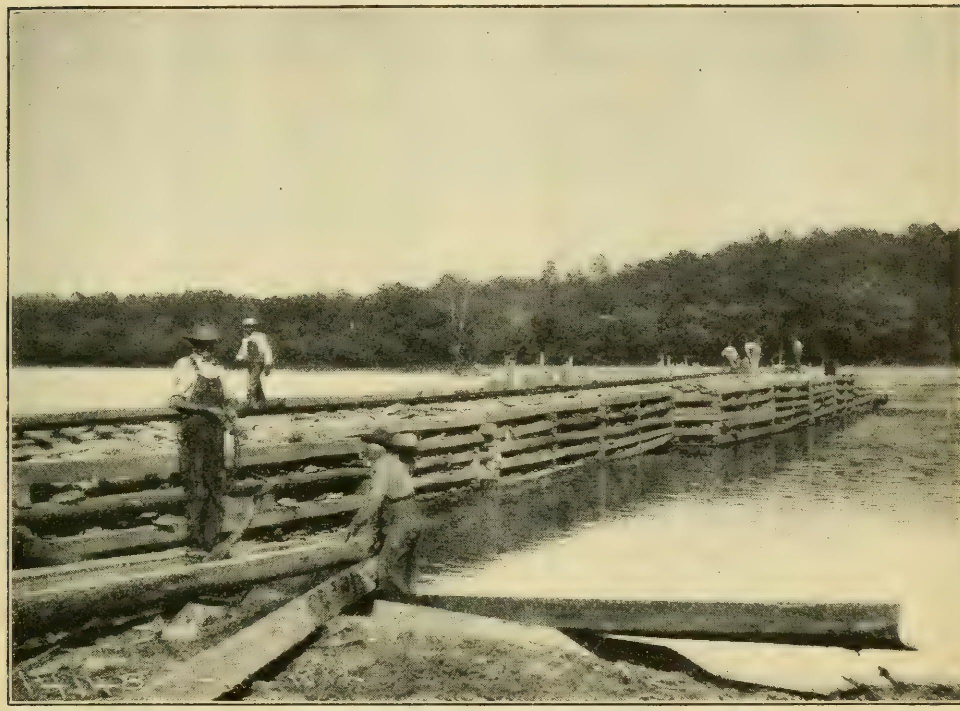


FIG. 25.—COFFER-DAM NO. 1, SHORTLY AFTER INAUGURATION OF WORK AT LOCK NO. 12.

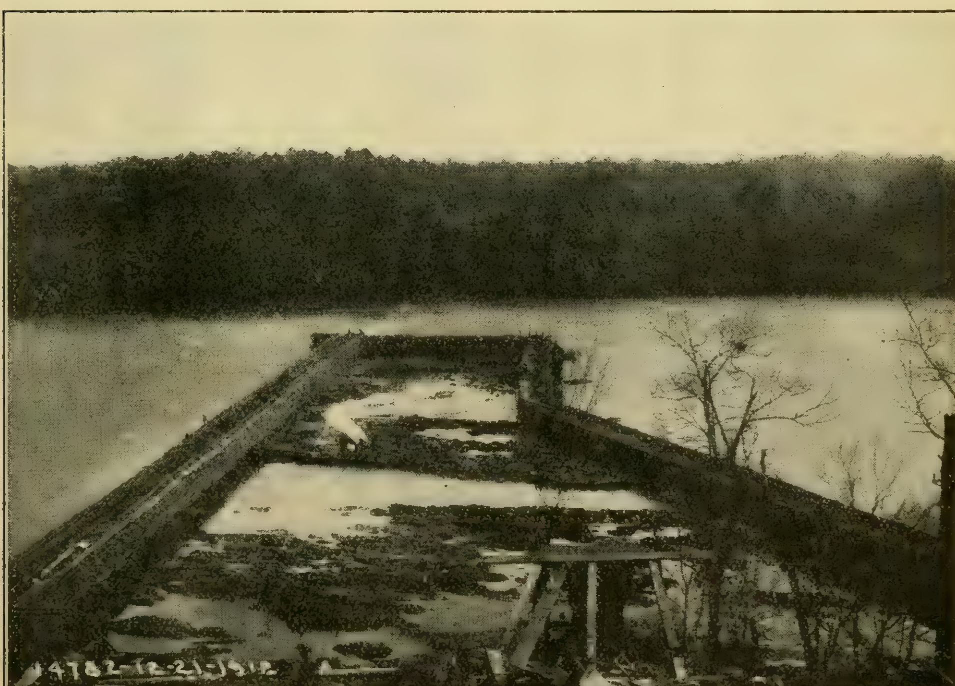


FIG. 26.—COFFER-DAM NO. 1 COMPLETED. DRILLING OF FOUNDATION STARTED.

stream, was made 30 ft. wide with three 10-ft. compartments. All divisions between compartments were carefully sheeted, as well as the outside of the cribs. The interior 10-ft. compartment was filled with clay and the two outer ones with stone.

In Coffer No. 1, the plan of dividing the inside of the crib roughly into two sections, filling the lower, down-stream section with stone and the upper one with clay, is not to be recommended as the most efficient. The cribs should be filled entirely with stone, and all stoppage of leakage should be made with a substantial earth or clay toe-fill on the up-stream side, care being taken to sheet carefully this up-stream face. When the toe-fill has been properly made, if there is much current, a 2- or 3-ft. layer of rough rip-rapping should be dumped on the outside to protect it from wash. If there is much wash of the earth toe-fill while it is being placed, it can be stopped effectively by building small stone jetties up stream 30 or 40 ft., at right angles to the line of the cribs, at intervals of 150 ft. or more. This scheme was used here with excellent results.

In the case in question, the first flood that topped the coffers took out a large portion of this inside fill, and the space formerly occupied by the earth had to be refilled with stone. It was also found that this interior fill was not effective as a leak stopper unless it was made like the outer river end, that is, in a tightly sheeted interior chamber which absolutely prevented any wash through it.

The first timber used in making the coffer-dam was squared on four sides, but, later, timber squared on two sides was used, which decreased the time of handling and expense at the saw-mills. All coffer-dam timber was hauled from the mills to the dam—about ½ mile—by teams. On Figs. 25 and 26 is shown the beginning of construction of this coffer and its completion. Fig. 27 shows typical sections of the various coffer-dams.

Pumps were started on December 3d, 1912, but numerous leaks developed under the down-stream coffer, and the pumps were shut down until the toe-fill could be fully placed along the south section. A portion of the river bed was finally unwatered by December 13th, 1912. Coffer No. 1 was entirely completed by December 21st, its total length being 1 640 ft. It contained 525 000 ft. b.m. of timber, and took a little more than 3 months to complete. The

filling for this coffer, as well as for all others, was carried in 1½-yd. cars, which were filled by hand, from quarries on each bank of the river, with stone and earth from earth pits in the bottom, and hauled in trains of 4 cars each, by a mule, out on the coffer as far as it was completed. This method proved satisfactory, and no great trouble was experienced.

On May 20th, 1913, a coffer-dam, 10 ft. wide, was started below Coffer No. 1 to encircle the tail-race excavation (see Figs. 23 and 27). As the water was very low at this time, it was not necessary

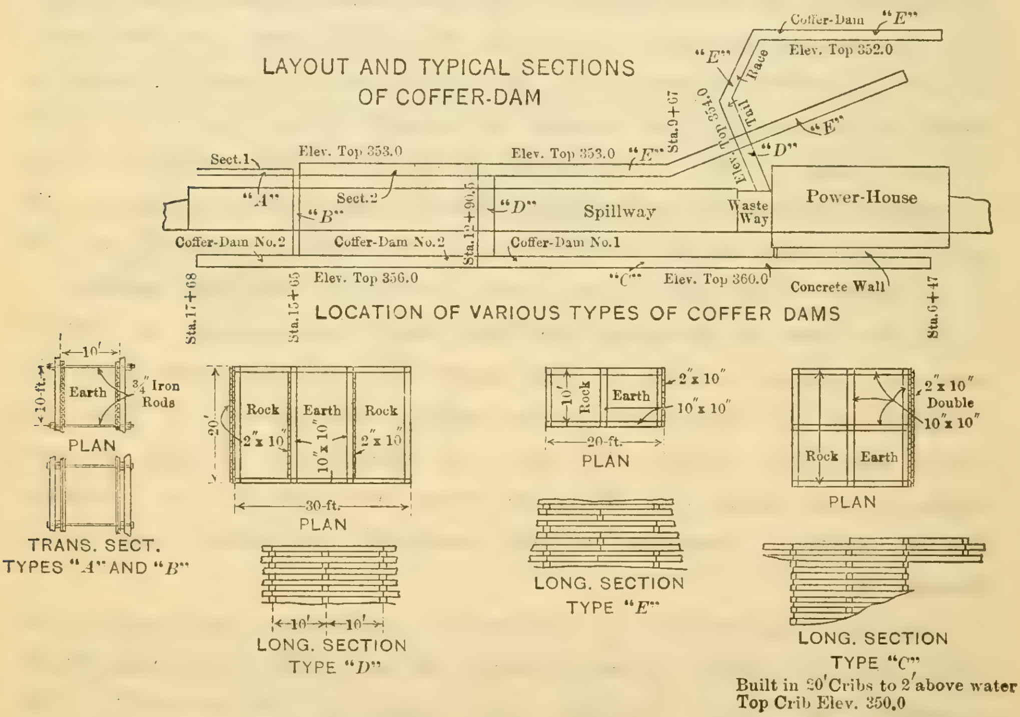


FIG. 27.

to build cribs on the shore, and they were constructed in place, the water being about 4 ft. deep. The average height of this coffer was 6 ft., the elevation of the top being 350.5 and its length 655 ft.

Coffer No. 2 was begun on June 17th, 1913, and the up-stream side was built of cribs, of the same size and in the same manner, as Coffer No. 1. A false deck was built above the water on this coffer and loaded with stone, allowing the water to run through the cribs until the up-stream section was practically completed and connection made with Coffer No. 1, when it was filled entirely with stone. This was done to keep the current as slow as possible to

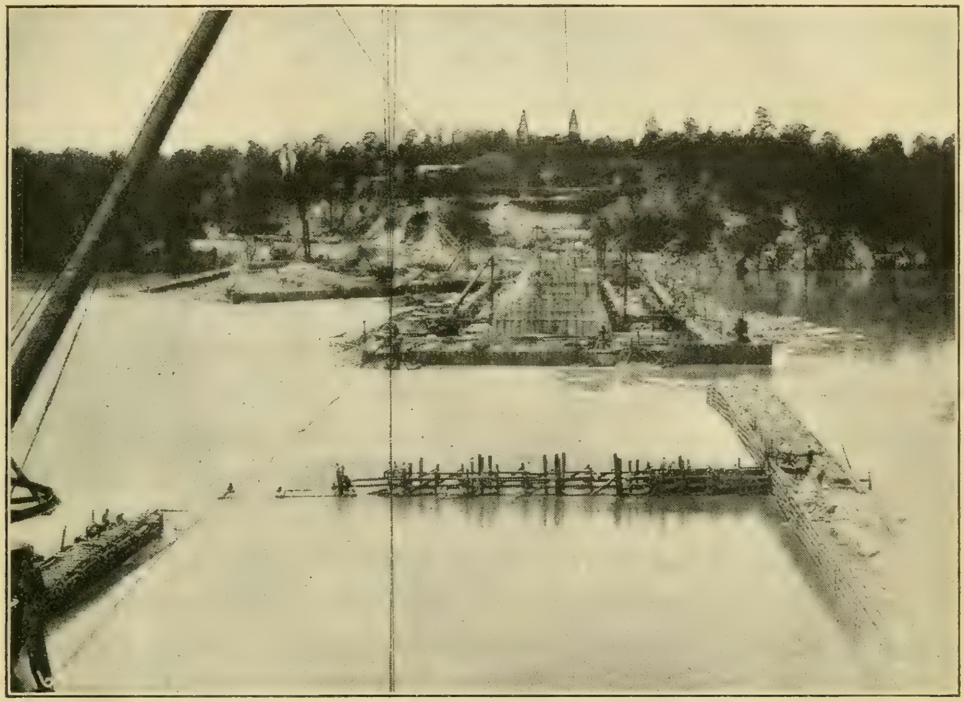


Fig. 28.—Second Coffer-Dam, Showing Construction of Light Partition Ribs Under Way.

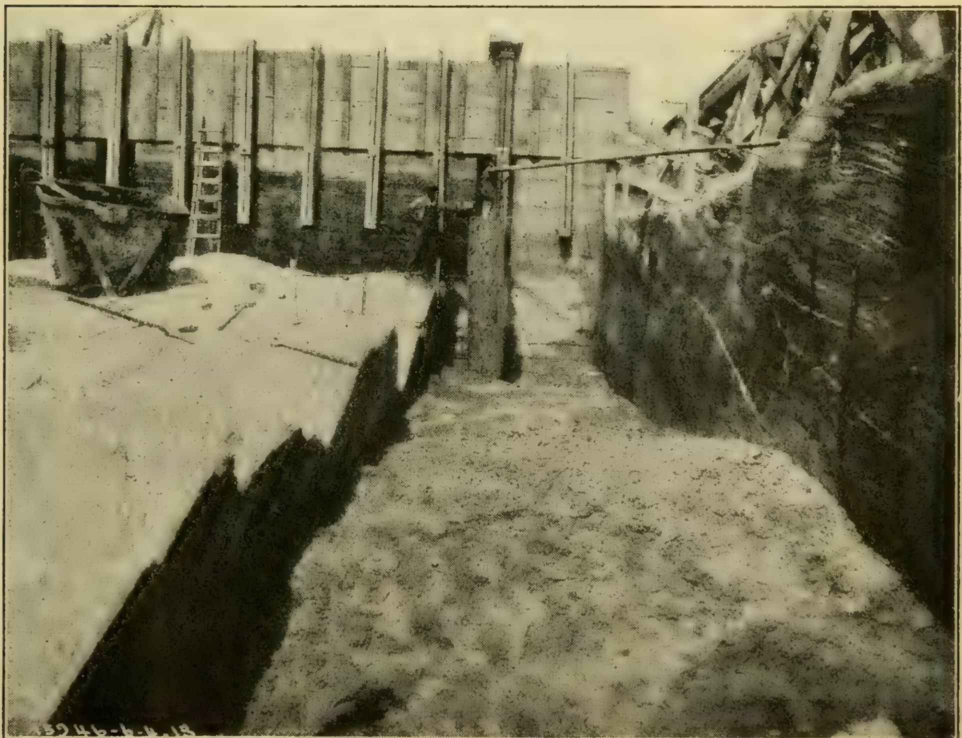


Fig. 29.—Cut-Off Trench Partly Filled with Concrete, Showing Type of Rock Foundations.

avoid backing up the water and deepening it before all the cribs were placed. It was much more difficult to put these cribs in place than those of Coffer No. 1, owing to the increased current in the river and the greater depth. The down-stream section was built by using two 10 by 10-in. uprights with two 10 by 10-in. wales on the inside and 2-in. sheeting nailed on the inside of the wales, with only a loam fill. This makes a very cheap and quickly constructed coffer-dam, where there is not much head and the water is slack and quiet, but it is not very stable, and is easily washed away. This coffer-dam was partly constructed when high water came and delayed the completion and closure. A cut-off coffer-dam was constructed in the same manner as the down-stream one, about half-way between the shore and Coffer No. 1, as shown by Fig. 28, permitting the work to be carried on in one-half of the coffer, while the remainder was being completed, thus advancing the work by about 10 days. The average elevation of the up-stream side of Coffer No. 2 was 354, and that of the down-stream side, 351.5.

Closure of the north line of cribs of Coffer No. 2 with old Coffer No. 1 was made on August 20th, 1913. The closure in the south line was made about August 27th, 1913.

The total length of all the coffer-dams was 3 423 ft., and the total quantity of lumber and timber used was 898 984 ft. b.m.

Excavation in Dam and Power-House Foundations.—In general, the rock throughout all the foundation of the dam and power-house was a hard, blue, thin-bedded slate with numerous quartz seams running through it. Its outcrop crossed the axis of the dam at an angle of about 45° and dipped down stream, thus affording additional security from leakage and sliding. The general rough character of this bottom is shown by Fig. 29. Occasional soft streaks, from 6 to 8 ft. deep and from 25 to 75 ft. wide, were encountered, and these were worked entirely out, down to the hard blue slate. All rock excavation was drilled with Ingersoll, E 24 drills, operated by air. These drills made 2-in. holes, having a depth which varied according to the character of the rock. This feature was watched carefully by the engineers on the dam, and the depth of hole to be drilled in various sections was given to the drill foreman; the quantity of powder to be used was also specified. The quantity of dynamite varied from 1½ to 6 sticks of 40% Red Cross du Pont

dynamite per hole. In wet holes, Forcite gelatine was used. As a rule, the holes were placed on the corners of 3-ft. squares.

Only two bad mud seams of any size were encountered, one in each abutment. The excavation was carried back into the hill in each case until these seams pinched out and good rock showed. Some small seams were encountered, and these were cleaned out thoroughly and grouted under pressure. The cut-off trench was carried from 6 to 15 ft. below the general level of the excavated bottom; its excavation is described under the heading "Channeling." No differences in the character of the rock in the "cut-off" trench from that in the bottom, already described, were found, except that the rock was somewhat more compact in the bottom of the cut-off trench.

After the drilling, blasting, and excavation of the materials was completed, and the bottom was fairly clean and compact looking, a gang of 20 men was put in on a 75-ft. section and generally worked there 2 days, barring, wedging, and picking the surface of the work. This was continued until all loose slabs of slate were removed, and then scrubbing, washing, and cleaning were continued until the foundation was absolutely clean and free from all loose materials. Wire brooms were used all over the bottom, and a 2-in. fire hose, with a ½-in. nozzle, under considerable pressure, was used in washing and scrubbing. If, after cleaning the bottom, there still appeared to be pockets or layers of soft material, it was again drilled, shot, and excavated, and then finally cleaned up. Wherever leaks of any consequence occurred (which were generally low down in the "cut-off" trench), a cone of spalls and mortar was built, completely enclosing the leak, the water being led out of a 1½-in. pipe from the bottom of the cone and another pipe rising out of its top. When the cone had set, the drain from the side was plugged and the water was allowed to rise in the vertical pipe. This pipe was brought up until some 15 or 20 ft. of concrete lay over the leak, and then it was grouted under pressure up to 50 lb. Where several bad leaks were encountered, a well was formed in a suitable place in the "cut-off" trench, and all the surrounding leaks were led to it through 1½ and 2-in. pipes. A pump was set up nearby to keep the water lowered in this well until the concrete was well set. The well was brought up to such height as was necessary to bring the water level to a stand, then it was pumped

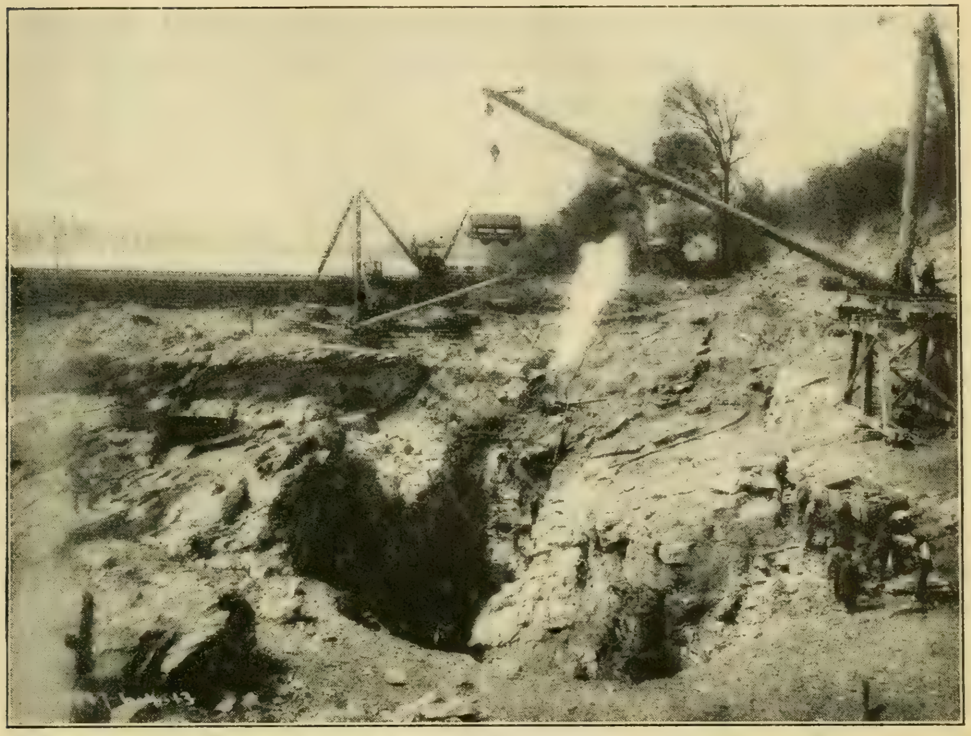


FIG. 30.—DEEP GRAVEL POCKET ENCOUNTERED IN FOUNDATION, BETWEEN UNITS NOS. 1 AND 2 OF POWER-HOUSE.

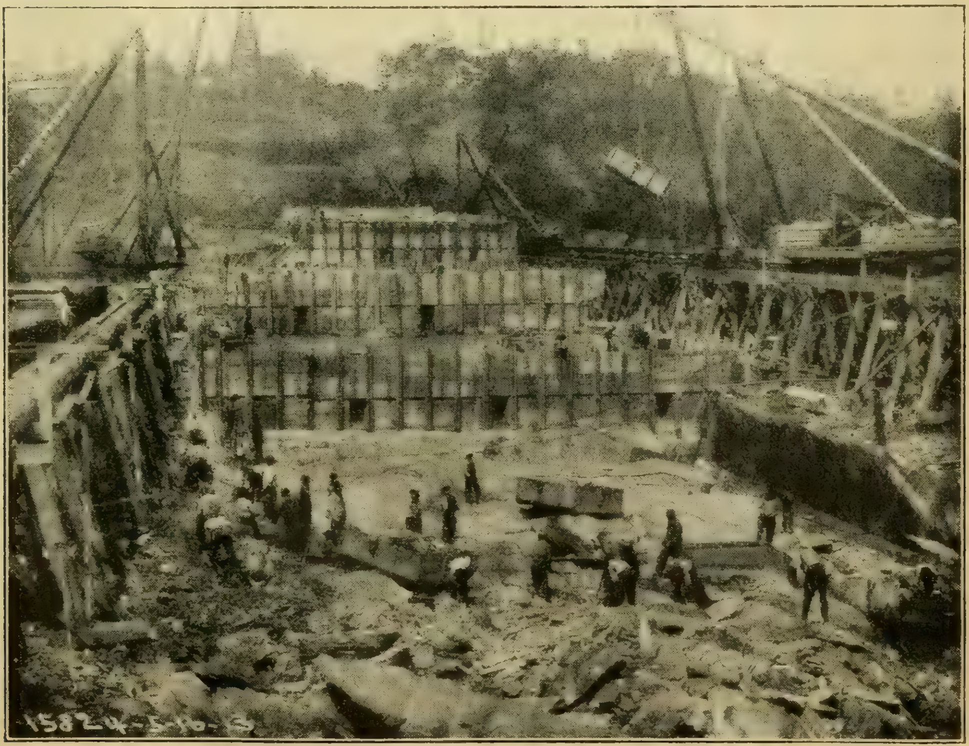


FIG. 31.—GENERAL VIEW, SHOWING METHOD OF HANDLING WORK AND TYPE OF WALL FORMS USED.

down, the forms and dirt were cleaned out of it, and grout pipes were brought up from the drains leading into the wall, and then filled with concrete. After the concrete had set, the leaks were grouted back, under pressure, through the original drain pipes. There were no serious leaks in any portion of the work.

The rock under the power-house was slightly heavier and more dense than in other portions of the foundations, but at the west end, between Units 1 and 2, and partly under Unit 1, a large cavity filled with sand, gravel, and slate boulders was found. This cavity extended the full width of the power-house up and down stream, and was about 50 ft. wide on top, narrowing to about 15 ft. at the bottom. It was wedge-shaped, with the small end down stream. Fig. 30 is a view of this hole. The lowest part of the hole was at Elevation 291.0, or 35 ft. below the bottom of the draft-tubes. The hole itself, the writers believe, was of the nature of an old "geological mill," and was worn out by the whirling around of gravel and sand in pot-holes, these pot-holes wearing eventually into each other. The hole was cleaned to the bottom, as a dentist cleans a cavity in a tooth, and then filled with concrete. The hole extended both up and down stream beyond the limits of the excavation, and may have been of considerable length. Two derricks were used in excavating it, handling $1\frac{1}{2}$-yd. cars from the hole to a narrow-gauge track, the material excavated being used for a toe-fill on the outside of the coffer-dam. After solid rock was reached at the bottom, 16-ft. holes were drilled in various places to make sure that there were no caverns below, but the rock drilled solid and clean. The 2 200 cu. yd. of concrete necessary to fill this hole to the general level of the surrounding bottom was poured through a chute, which was built directly from the mixer.

The following quantities were excavated:

Dam	Earth	3 379 cu. yd.
	Rock	15 442 " "
Power-house	Earth	8 572 " "
	Rock	11 818 " "
Tail-race	Earth	8 029 " "
	Rock	11 245 " "
Total		54 845 cu. yd.

Excavation was started on the dam and power-house foundations on December 17th, 1912, and completed on October 20th, 1913.

Channeling the Cut-Off Trench.—A cut-off trench, 10 ft. wide, from 6 to 15 ft. below the general bottom of the dam, extending under both the dam and power-house, and well keyed into both banks of the river, was cut. Two channelers were used for both sides of a large portion of this trench.

Time was lost at the beginning of the work in starting these channelers, and it was seen that they would have to be supplemented in order that the excavation of the trench might keep up with that of the general bottom, and be ahead of the concreting. Two quarry bars were purchased and two Ingersoll, E 24 drills were set up on each, channeling in the power-house foundations. These drills put down holes 6 in. apart on the line of the trench. This method was not satisfactory, as the rock was badly shattered in blasting and the trench had to be carried much deeper than was necessary in order to get below the shattered rock. The channelers gave a clean cut and much better job all around, and are far superior to the drill work.

In the abutments only the up-stream face of the abutment was channeled by drills, and no actual cut-off trench was made, the whole excavation into the hill being practically such a trench. The thickness of the channeler steel varied from $1\frac{1}{4}$ to $2\frac{1}{2}$ in.

Throughout the work on the cut-off trench, 40% Red Cross dynamite was used, with the exception of a few wet holes where 40% Forcite was used.

Disposal of Waste Materials.—At the beginning of the work, before the derricks were in position to handle the excavation, all excavated material was moved by cableways, in 4-yd. steel skips, to 7-yd. dump cars hauled by an engine on the Low-Level Line, the material being used to widen the embankment for that line. This method, however, was very slow and unsatisfactory, it being hard to stop and dump a heavy skip over a 7-yd. car. Later, the skips were landed on flat-cars, taken about 1 000 ft. up the Low-Level Line, on a steep hillside, where a large guy derrick was erected to pick them up and unload them. A large part of the draft-tube excavation in the power-house was made in this way. When the traveling derricks were put in service on the up-stream coffer-dam, a track was laid on the coffer-dam and a dinky locomotive, handling 7-yd. cars, took the excavation from

these derricks. It was lifted out of the bottom in 4-yd., steel skips and dumped on a slanting platform built for the purpose. The material was then dumped over the side of a 10-ft. bank into the river about 200 ft. up stream from the dam. Later, these derricks landed loaded skips on flat cars, handled by a dinky engine and a locomotive crane, on the dump already mentioned, picked them off and wasted them down the bank into the river. Material in the excavation, out of reach of the up-stream derricks, was picked up by those traveling on the down-stream coffer-dam and passed across to the up-stream derricks which, in turn, disposed of it as described.

The tail-race excavation was handled and disposed of in a different way: Two traveling derricks started at the east side of the excavation and backed off toward the west as they cleaned up the excavation. The bottom was drilled, shot, and then placed by hand in 4-yd. skips. These were picked up by the derricks and discharged into 7-yd. dump-cars, which were hauled out of the bottom by a cable operated by a regular hoisting engine, and disposed of down stream about 500 ft. from the side, on an embankment. All the tail-race excavation was made in this manner very quickly and satisfactorily.

In general, the following criticism is offered of the methods used on this work: A cableway is not satisfactory in handling the excavated materials, as it is not only expensive, but slow. A better scheme for the work in Coffer-dam No. 1 would have been a system of standard-gauge tracks laid down from the west bank of the river, through the tail-race, to its extreme east end. The excavation should then have started at that end, backing off toward the west, the spoil, in skips, being lifted with the derricks and discharged into 7-yd. dump-cars, hauled by dinky engines. The power-house excavation could have progressed rapidly, also, at the same time, and both cableways would have been free to erect derricks, get plant and timber out to the job, and handle concrete, thereby advancing the work greatly. The writers are of the opinion that, as a rule, the cableway should be restricted to moving plant, derricks, forms, and general work, and also such concreting as is advantageous to place with it, and that it should not be made the main method for the transportation of all materials on the job, as it was here.

Forms Used for the Dam.—The forms used on the vertical faces of the dam were of the cantilever type, built in sections, 12 ft. long

and 6 ft. high, with upright posts extending 6 ft. below the bottom of the sheeting. Figs. 29 and 31 give a general idea of these forms. All the sheeting was sized and dressed to $1\frac{3}{4}$ in. in thickness, and all vertical posts were made of two 4 by 8-in. timbers nailed together, with 1-in. blocks between them at the top, middle, and bottom. The waling pieces were 4 by 8-in., or 6 by 8-in., laid flat. These forms were built complete and raised in one piece, from one lift to another, by a 3-ton chain block hung from an A-frame, guyed back with form wire to hook-bolts in the concrete. This particular kind of form was not used throughout the job, for, as the work progressed and carpenters became rather scarce, sections of this type were abandoned, and 3 by 6-ft. panels were made in the carpenter shop and simply tacked to the posts. This type was practically the same as the first and was handled more quickly and easily than the forms of heavier section.

The following method was used to hold the forms in place: When a form was completed, $\frac{3}{4}$-in. bolts, 22 in. long, with an ogee washer on one end, were placed in them, 10 in. below the top, the bolts having a 10-in. hold in the concrete when it was poured around them. Also, when the pour in the form was completed, and while the concrete was still soft, $\frac{3}{4}$-in. bolts, 12 in. long, with a hook on one end, were set in the concrete opposite each post and about 6 ft. back from the face of the form. When a section form or panel form was raised, it was lifted until the bottom of the sheeting was above the bolts set 10 in. below the top of the previous pour of concrete. The posts were fitted on these bolts which held the form in place, passing through the slot between the two 4 by 8-in. posts. A waling piece at the top of each section was used to wire the form back to the hook-bolts placed in the concrete. Wedges were placed at the bottom of the post to line up the form. The panel forms were handled in the same manner, the loose posts being placed on the bolts previously set in the concrete, and the panels dropped into place behind them and tacked to the posts. As the speed of filling the forms increased, two walings were used instead of one. All expansion joint forms were held in this same way. For the curved section of the dam, on the down-stream face, ribs were cut to the proper radius in the form shop and brought down to the job where sheeting was nailed on them in place at the dam. The ribs for all curved forms were made of two 2 by 8-in. timbers nailed together, with a 1-in. spacing block between them, leaving a 1-in.

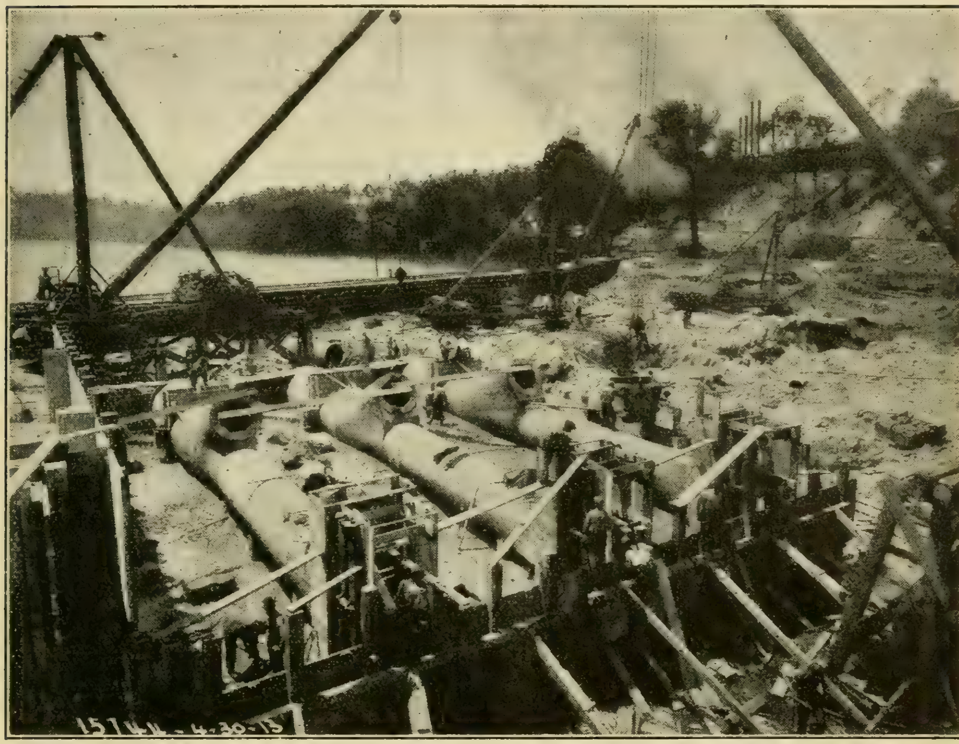


FIG. 32.—FORMS FOR WASTEWAY OR BLOW-OFF CULVERTS.

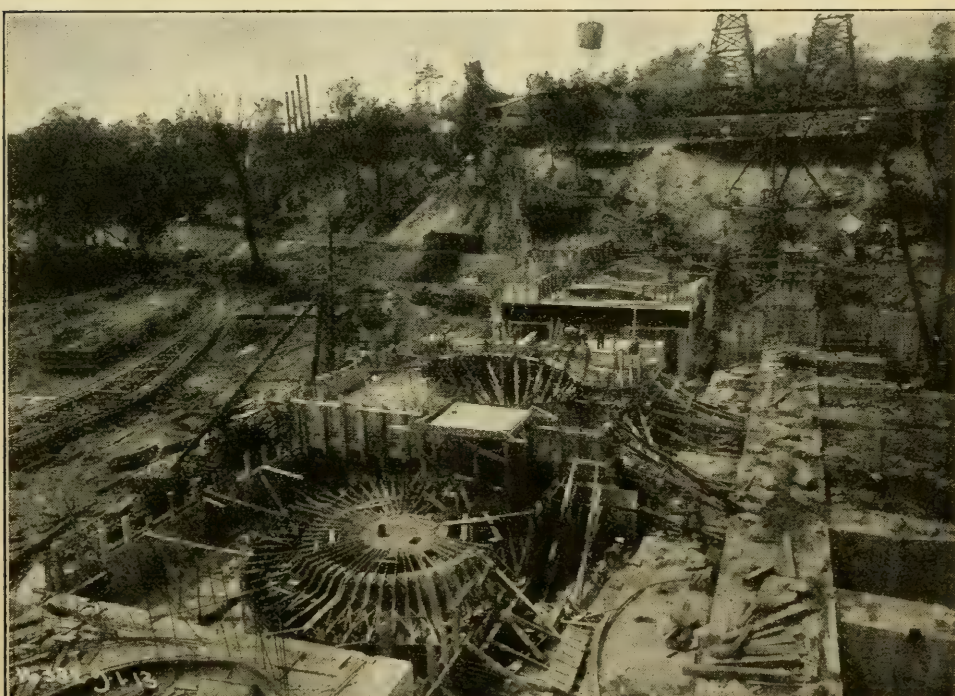


FIG. 33.—SCROLL CASING FORMS IN PLACE DURING CONCRETING OF FIRST STAGE OF CONSTRUCTION.

slot to catch the anchor-bolts, in the same manner as the vertical forms. In some of the big blocks on the dam, where the concrete came up slowly on the forms, two 2 by 8-in. pieces were used in place of the two 4 by 8-in. timbers, and proved satisfactory.

As a general rule, these forms worked well and were fairly economical of timber. However, it is not the fastest method; after a form is poured, sufficient time has to be allowed for the concrete to set properly and for the bolts to take hold. More rapid work could be done with a form of the same general type, but built up continuously, releasing the bottom sections as the opportunity arose. In building the forms continuously, one could be raised as soon as the carpenters could walk on the concrete, and the next pour could be made from 12 to 24 hours sooner than with the type used. The latter, however, would take considerably more timber, but the speed would be increased, and in a country where timber is very cheap and speed a big consideration, the continuous type might pay. Fig. 32 shows the type of forms used in moulding the wasteway culverts in the section of the dam adjoining the power-house.

Forms Used for Power-House.—All straight and vertical wall forms in the power-house were of the same type, and were handled in exactly the same manner as the dam forms.

All special or curved forms, such as for draft-tubes, scroll casings, and penstocks, were made in the form shop according to detailed drawings furnished by the Company's engineers.

The type of scroll casing used (Fig. 10), was more or less of an innovation, and there was considerable discussion at the beginning of the work as to the practicability of moulding such a complicated opening. The form work, as can be readily seen from Figs. 34, 35, and 36, was highly complicated, and, even when carefully detailed, was difficult of execution in the field, for several reasons: First, the average carpenter foreman is out of his depth on a complicated drawing of this kind; secondly, the setting and lining up of a large form of this character is difficult of execution; thirdly, unless the laying out of the forms is done carefully, they are hard to get out. In the case at Lock 12, the engineers made all the calculations and furnished a sufficient number of detailed drawings to enable the work to be laid out and assembled in the form shop. A competent engineer assisted the carpenter foreman in reading these drawings properly, and laying

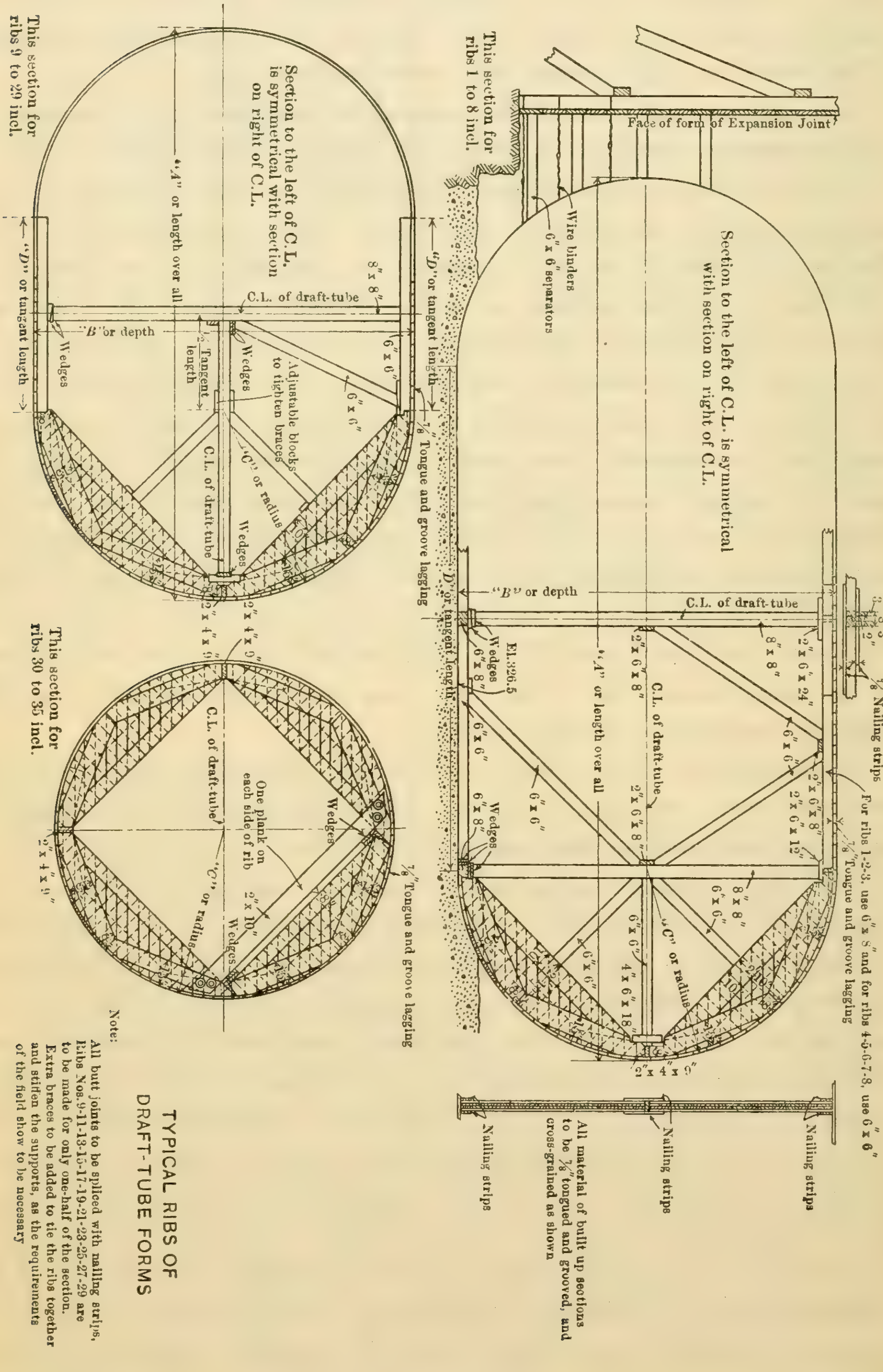

TYPICAL RIBS OF DRAFT-TUBE FORMS
This section for ribs 1 to 8 incl.
This section for ribs 9 to 29 incl.
This section for ribs 30 to 35 incl.
Face of form of Expansion Joint
Section to the left of C.L. is symmetrical with section on right of C.L.
Section to the left of C.L. is symmetrical with section on right of C.L.
Wire binders
6" x 6" separators
C.L. of draft-tube
"A" or length over all
"B" or depth
"C" or radius
"D" or tangent length
½ Tangent length
Adjustable blocks to tighten braces
Wedges
⅞" Tongue and groove lagging
⅞" Nailing strips
For ribs 1-2-3, use 6" x 8" and for ribs 4-5-6-7-8, use 6" x 6"
El. 326.5
One plank on each side of rib
Nailing strips
All material of built up sections to be ⅞" tongued and grooved, and cross-grained as shown
Note:
All butt joints to be spliced with nailing strips.
Ribs Nos. 9-11-13-15-17-19-21-23-25-27-29 are to be made for only one-half of the section.
Extra braces to be added to tie the ribs together and stiffen the supports, as the requirements of the field show to be necessary

FIG. 34.

out the various parts of the form. The greatest care was exercised in setting up the forms and holding them in place. The work was handled in this manner with no great difficulty, and the final results were excellent.

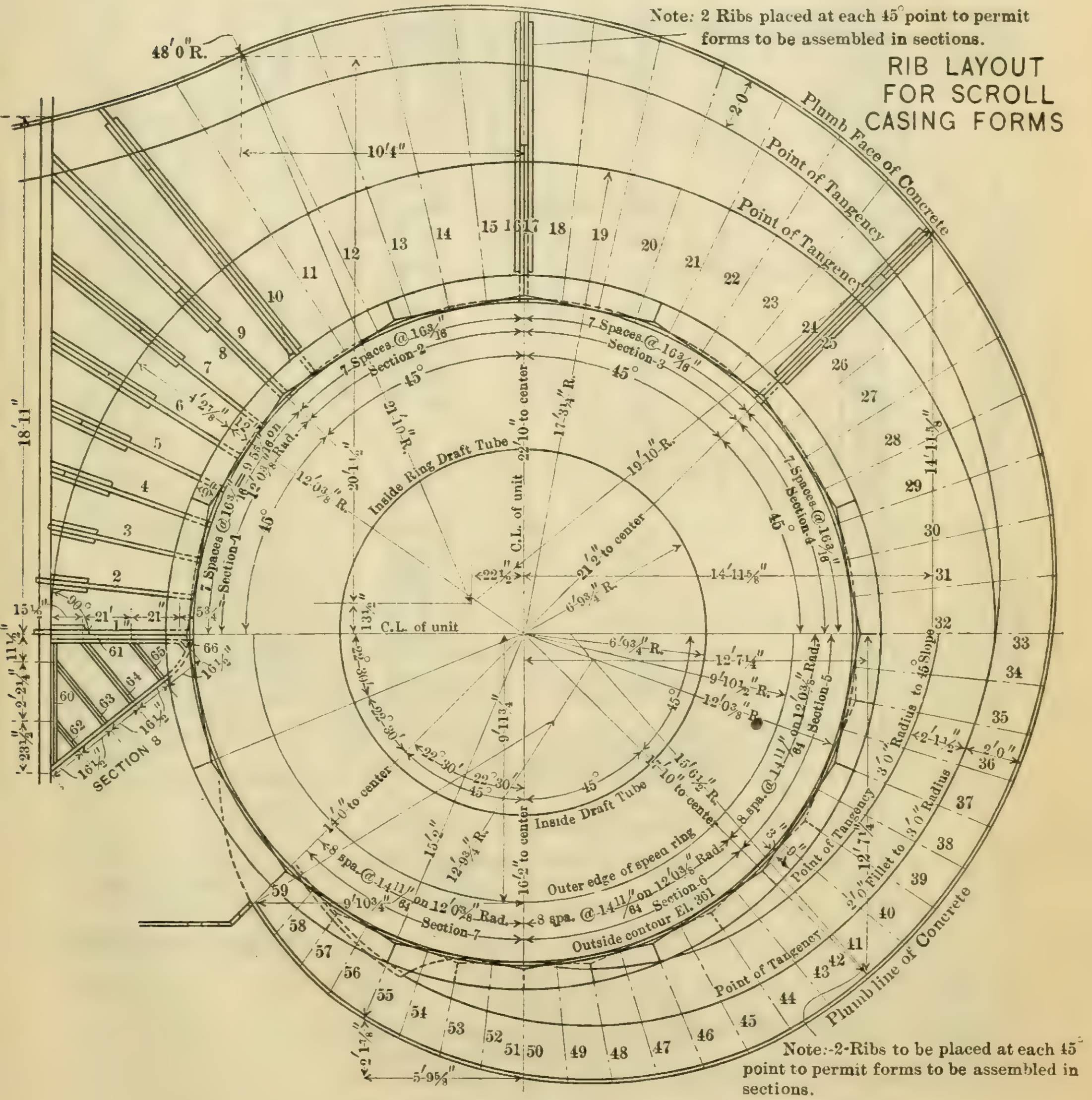


FIG. 35.

In detail, these complicated forms were built as follows: All the ribs of the scroll casings (Figs. 35 and 37), were made according to the drawings furnished, and were then set up in the shop in their proper positions, as laid out, and sheeted with 1-in. material. The

detailed dimensions for the ribs are given in Table 6. On the curved portions, 1 by 1-in. lagging was used, in order that it might bend easily and conform to its intended shape. The whole scroll casing was built in the shop in eight sections in order to facilitate handling

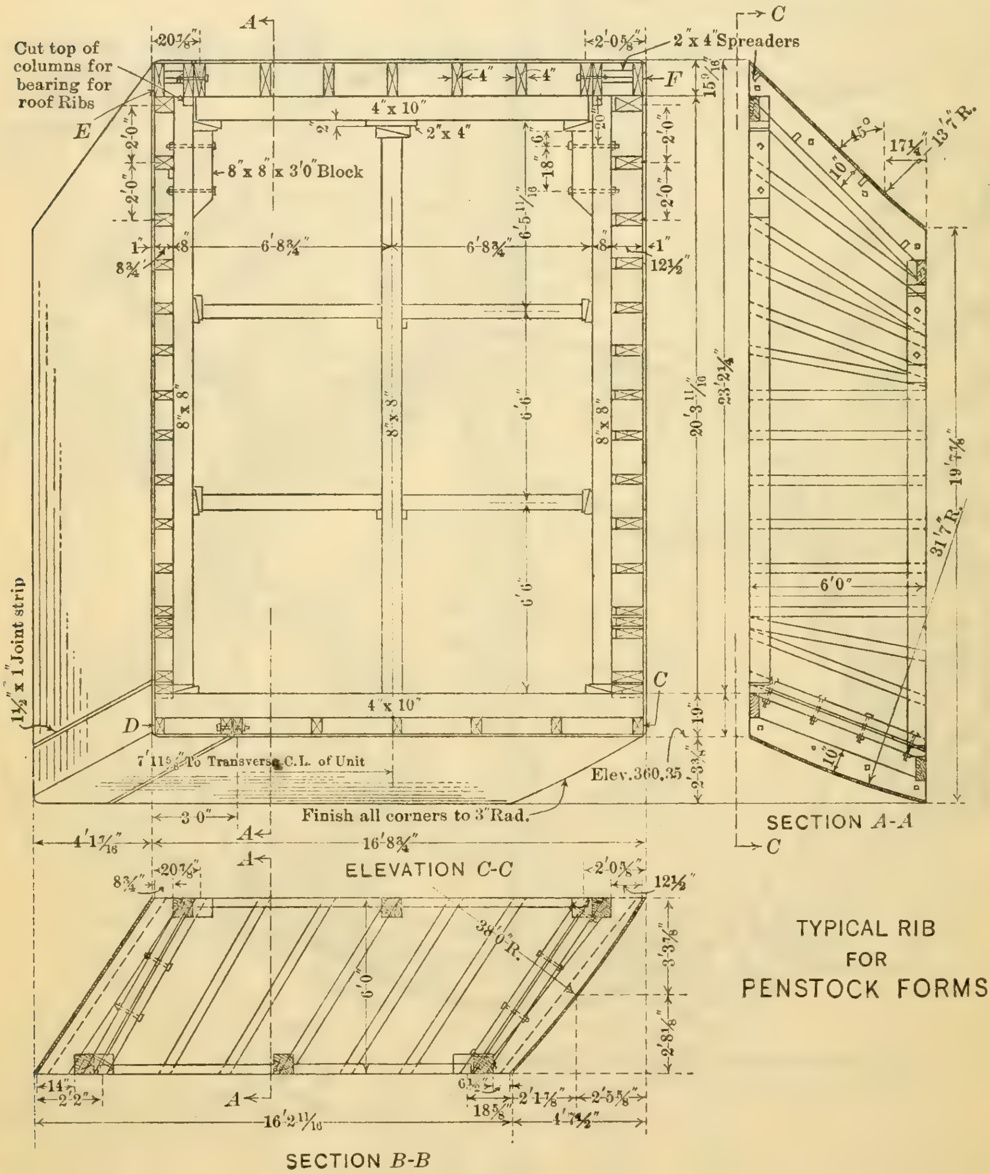


FIG. 36.

and erecting, each section being complete in itself. For the draft-tubes (Figs. 34 and 38), all the ribs were made and fitted together on the form-shop floor; they were then taken down and hauled to the dam and re-assembled on a platform from which a derrick could lift and

TABLE 6.—DETAILED DIMENSIONS FOR RIBS OF SCROLL CASING FORMS.

Rib. No.	*A*		*B*		*C*		*D*		Rib No.	*a*		*b*		*c*	
	Ft.	In.	Ft.	In.	Ft.	In.	Ft.	In.		Ft.	In.	Ft.	In.	Ft.	In.
10	2	2½	5	4¼	0	6	12	9⅝*	33	2	0	5	3⅝	4	0
11	2	1	4	7½	0	6	11	11⅜*	34	2	0	5	0¾	3	9½
12	2	0	4	4	0	6	11	6⅞	35	2	0	4	10	3	7½
13	2	0	4	2	0	6	11	4⅞	36	2	0	4	7¾	3	5¼
14	2	0	4	0	0	6	11	2⅞	37	2	0	4	5½	3	3¼
15	2	0	3	9½	0	6	11	0⅜	38	2	0	4	3	3	1
16	2	0	3	7	0	6	10	9⅞	39	2	0	4	1	2	11
17	2	0	3	7	0	6	10	9⅞	40	2	0	3	11¼	2	8¾
18	2	0	3	4½	0	6	10	7⅛	41	2	0	3	9⅝	2	6⅜
19	2	0	3	1¼	0	6	10	4⅛	42	2	0	3	9⅝	2	6⅜
20	2	0	2	10¼	0	6	10	1⅛	43	2	0	3	8	2	4
21	2	0	2	7¼	0	6	9	10⅛	44	2	0	3	6	2	2
22	2	0	2	4¼	0	6	9	7⅛	45	2	0	3	3½	1	11¾
23	2	0	2	1¼	0	6	9	4⅛	46	2	0	3	1¼	1	9¾
24	2	0	1	10½	0	6	9	1⅜	47	2	0	2	10½	1	7¼
25	2	0	1	10½	0	6	9	1⅜	48	2	0	2	8	1	4½
26	2	0	1	7¾	0	6	8	10⅝	49	2	0	2	5	1	1¾
27	2	0	1	4¾	0	6	8	7⅝	50	2	0	2	1⅝	0	10½
28	2	0	1	1½	0	6	8	4⅜	51	2	0	2	1⅝	0	10½
29	2	0	0	10¼	0	5⅛	8	1⅛	52	2	0	1	11	0	7½
30	2	0	0	6¾	0	3⅜	7	9⅝	53	2	0	1	8	0	4½
31	2	0	0	3¾	0	1⅞	7	6⅝	54	2	0	1	4¾	0	1½
32	2	0	0	0¾	0	0⅜	7	3⅝	55	2	0	1	1½	0	1¾
									56	2	0	0	10½	0	5
									57	2	0	0	7¼	0	8
									58	2	0	0	3½	0	11¾
									59	1	11½	0	0	1	3¾

* R. generates ellipse.

place them. A concrete cradle, containing a number of anchor-bolts, had previously been built, conforming exactly to the curve of the bottom of the draft-tube, and these ribs were placed on a 1-in. board resting on this cradle. Tongued and grooved 1-in. sheathing was nailed to the ribs after they had been properly lined and braced. Figs. 33 and 39 give an excellent idea of how these forms were constructed, and Figs. 34 and 38 show the details. The tops of the draft-tube forms were covered with 2 by 6-in. planks which extended out and helped support and keep in place the bottom section of the scroll case above. When a scroll case was first put in place, it was blocked up on wooden posts to exact line and grade and then small 1 by 3-ft. concrete piers were poured up under the joints and the middle of each section. These piers also contained anchor-bolts, and after the concrete of the piers had set, the forms were bolted tightly to them. They were also wired down to hook-bolts set in the concrete below, to keep the form from floating. Penstock forms, Fig. 36, were made in sections, 6 ft. long, in the carpenter shop; they were then knocked down and transported on cars to the dam, where they were re-assembled

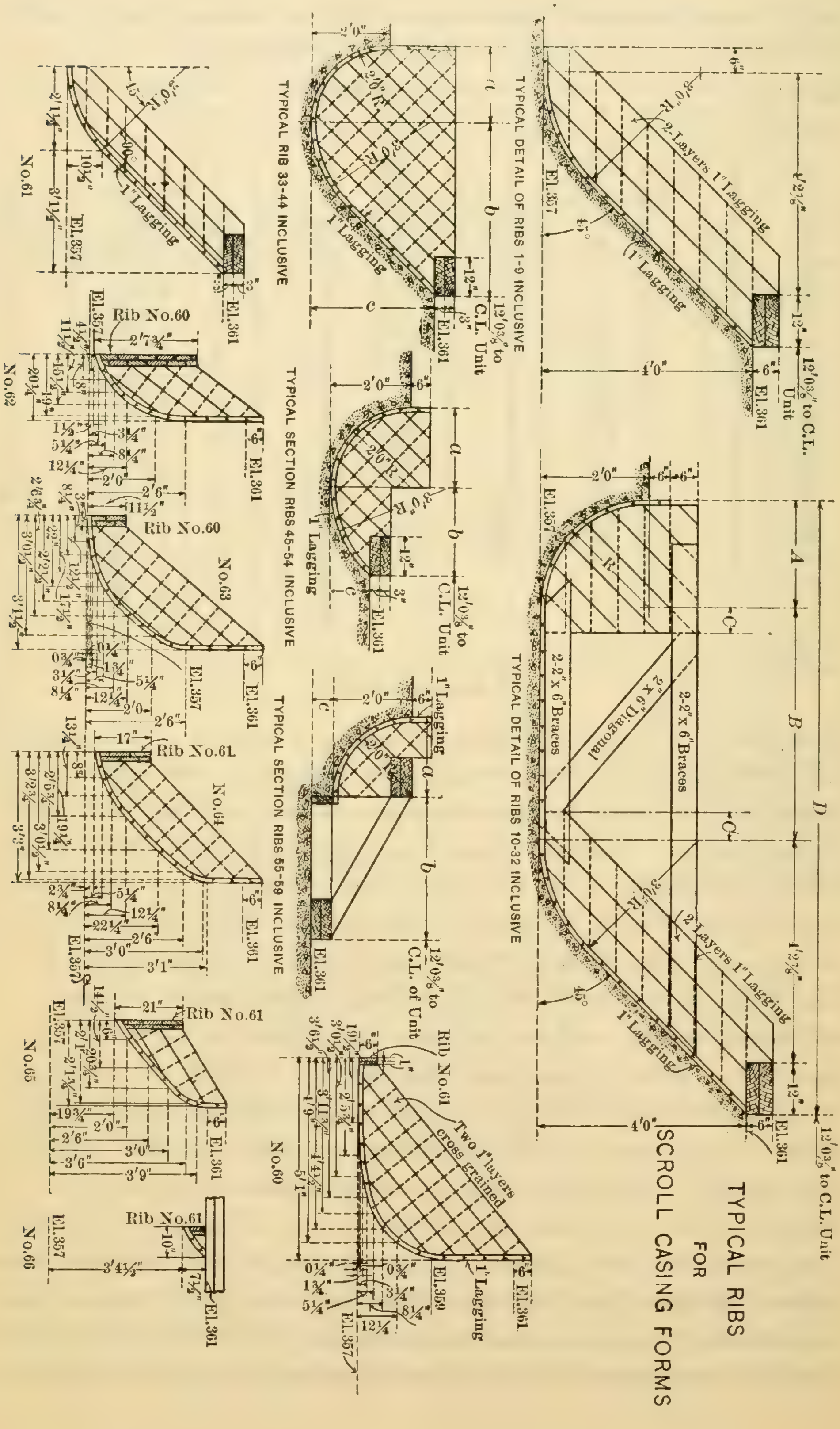
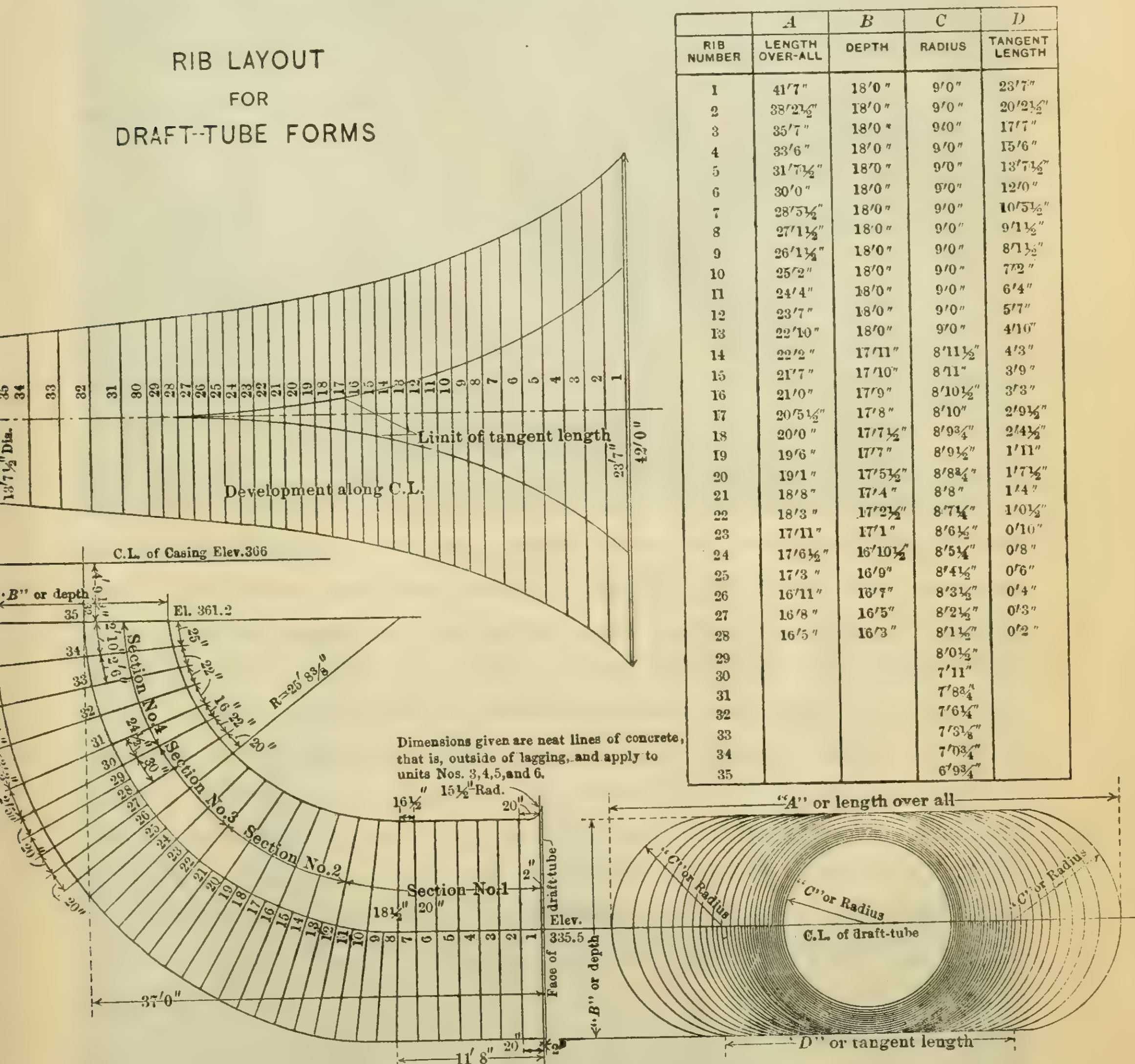

TYPICAL RIBS
FOR
SCROLL CASING FORMS
TYPICAL DETAIL OF RIBS 1-9 INCLUSIVE
TYPICAL DETAIL OF RIBS 10-32 INCLUSIVE
TYPICAL RIB 33-44 INCLUSIVE
TYPICAL SECTION RIBS 45-54 INCLUSIVE
TYPICAL SECTION RIBS 55-59 INCLUSIVE
2-Layers 1" Lagging
1" Lagging
2-2"x 6" Braces
2"x 6" Diagonal
Two 1" layers cross grained
12'0 3/8" to C.L. Unit
El. 357
El. 361
No.60
No.61
No.62
No.63
No.64
No.65
No.66
Rib No.60
Rib No.61

FIG. 37

on a platform adjoining a derrick which could pick them up and place them. A double cradle with anchor-bolts in it was prepared beforehand, in the same manner as the cradle for the draft-tubes. Types of drawings from which these forms were built are Figs. 34, 36, and 37.

The system of building complicated curved forms on a large laying-



	A	B	C	D
RIB NUMBER	LENGTH OVER-ALL	DEPTH	RADIUS	TANGENT LENGTH
1	41'7"	18'0"	9'0"	23'7"
2	38'2½"	18'0"	9'0"	20'2½"
3	35'7"	18'0"	9'0"	17'7"
4	33'6"	18'0"	9'0"	15'6"
5	31'7½"	18'0"	9'0"	13'7½"
6	30'0"	18'0"	9'0"	12'0"
7	28'5½"	18'0"	9'0"	10'5½"
8	27'1½"	18'0"	9'0"	9'1½"
9	26'1½"	18'0"	9'0"	8'1½"
10	25'2"	18'0"	9'0"	7'2"
11	24'4"	18'0"	9'0"	6'4"
12	23'7"	18'0"	9'0"	5'7"
13	22'10"	18'0"	9'0"	4'10"
14	22'2"	17'11"	8'11½"	4'3"
15	21'7"	17'10"	8'11"	3'9"
16	21'0"	17'9"	8'10½"	3'3"
17	20'5½"	17'8"	8'10"	2'9½"
18	20'0"	17'7½"	8'9¾"	2'4½"
19	19'6"	17'7"	8'9½"	1'11"
20	19'1"	17'5½"	8'8¾"	1'7½"
21	18'8"	17'4"	8'8"	1'4"
22	18'3"	17'2½"	8'7¼"	1'0½"
23	17'11"	17'1"	8'6½"	0'10"
24	17'6½"	16'10½"	8'5¼"	0'8"
25	17'3"	16'9"	8'4½"	0'6"
26	16'11"	16'7"	8'3½"	0'4"
27	16'8"	16'5"	8'2½"	0'3"
28	16'5"	16'3"	8'1½"	0'2"
29			8'0½"	
30			7'11"	
31			7'8¾"	
32			7'6¼"	
33			7'3⅛"	
34			7'0¾"	
35			6'9¾"	

Fig. 38.

out floor, and also all curved ribs for other forms at the carpenter shop, is recommended on similar work. The shop, however, as previously noted, should be much closer to the job than it was on this work. The concrete-cradle method of setting these large forms is recommended as the best and most accurate.

Quarrying Operations, Zion Quarry.—Fig. 24 shows the general layout of the quarry tracks and plant, and the general relation between the various portions of the work. The actual work of clearing and erecting the camp and plant at Zion Quarry was started on November 11th, 1912. On January 20th, 1913, a force was put to work stripping from the toe of the hill, and tripod drills were started. On January 20th, a well drill was also put to work on the side of the hill, drilling the first round of 40-ft. holes; the well-drill outfit was increased later to a total of three. On February 12th, 50 teams were put to work to strip the top of the quarry. On February 16th, 1913, the first large blast was made, and actual quarrying operations were started. The crushers had been started on February 24th, to crush run-of-the-quarry stone for ballast purposes on the railroad and also to get the new plant in good running order by the time materials were needed for concreting at the river. One locomotive was assigned to the quarry service, and did no other work than to spot cars to the derricks in the quarry and to the crushers.

Five steel guy derricks were erected along the face of the hill, on about 100-ft. centers, after the first round of well-drill holes had been shot, the toe of the hill having been previously shot out. These derricks handled 5-yd. steel skips to the points wanted in the quarry, where they were loaded by hand. Stone too large to be lifted by hand and too small for cyclopean masonry was broken with a "skull cracker," which consisted of a cast-iron ball weighing 1 ton and fixed to the hoist line of the derrick by a trigger tripped with a hand line from below. The ball was spotted over a stone carefully, hoisted about 50 ft. and dropped, smashing the stone effectually. Crusher muck was placed in four skips which were loaded by derricks on a flat car. These loaded flat cars were picked up from all the derricks by a locomotive, pulled out, and spotted at the washing platform, and a train of empties was placed at the derricks. After carefully washing all mud from the crusher muck, the cars were spotted to a large timber guy derrick which picked up the skips and dumped them directly into the No. 9 crusher. The dumping device was simply a heavy inclined platform, on which the skip was lowered and prevented from sliding down by a lug under the skip catching a lug on the back of the platform. The skip was held and the contents slid out. After passing through the crushers, the stone was

FIG. 39.—DRAFT-TUBE FORMS UNDER CONSTRUCTION, SHOWING CONCRETE CRADLE ON WHICH THEY WERE SET UP.

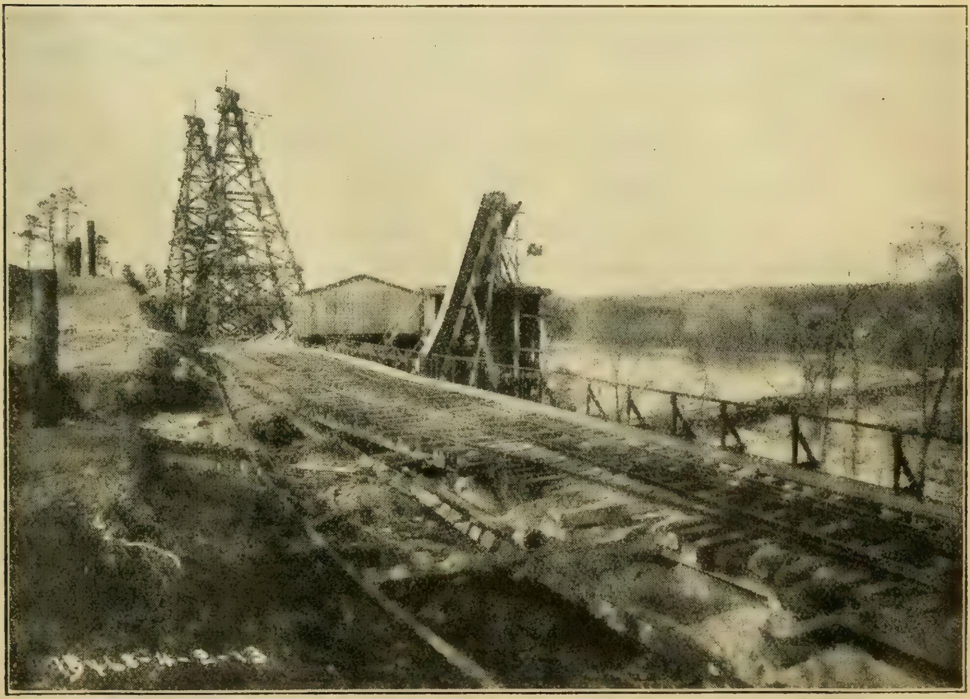


FIG. 40.—END OF HIGH-LEVEL LINE, SHOWING CABLEWAYS, CEMENT WAREHOUSE, AND MIXER BIN.

carried by a belt conveyor to a bin of 1 800-cu. yd. capacity, and drawn from this bin directly into hopper-bottom cars by gravity. (See Fig. 21.)

Well-drills were used for all the heavy work, being augmented in the bottom by tripod drills and Jap hammers. Well-drill holes varied in depth from 40 to 60 ft. These holes were drilled from 15 to 18 ft. back from the face and at about 18 to 20-ft. centers, 4 by 8-in. sticks of Forcite gelatine, 40% dynamite, being used. The largest size of Forcite obtainable was 4 in. in diameter, and the holes originally drilled to 6 in. were reduced to 5⅝ in. in order to get the full benefit of the explosive of this size. Only three rounds were taken from the face of the quarry at Zion owing to the heavy over-burden farther in the hill. After they had been cleaned up, about 6 ft. of earth, overlying the rock in a meadow directly in front of the quarry, was stripped back a distance of about 75 ft., and the quarry was then carried vertically downward 60 ft. for a total width of 100 ft. and a length of 450 ft. The seepage from a nearby creek was small and easily taken care of by a No. 6A, 8 by 5 by 13-in., 100-gal. pump. The stone was of excellent quality, and seemed to grow denser with the depth. A well-drill test hole indicated the thickness of the stone below the general surface of the ground to be more than 75 ft.

All crushing operations were done in daylight; and, while getting out the crusher muck during the day, the cyclopean stone was sorted as uncovered and stored in a pile alongside the derricks. At night the cyclopean stone was loaded on flat cars, and the night shift also carried on mucking and stripping operations. The quarry afforded only a small quantity of cyclopean stone, principally on account of the limited space and the necessity for getting out enough crushed stone to keep work going at the dam. It was not possible to take the time to get out the cyclopean stone, because by doing that the crushing would fall behind. In other words, the quarry was such that a sufficient quantity of either crusher muck or cyclopean stone could have been gotten out separately, but not both at the same time. Difficulties and uncertainties in the transportation of coarse aggregates from the outside would not allow that source to be depended on entirely, and reliance had to be put principally on the company's crushers. For these reasons the percentage of cyclopean stone at the dam was much smaller that it would have

been under more favorable circumstances. In this case, the percentage had to be sacrificed to other requirements which were considered more important.

Ellison Quarry.—Ellison Quarry, about 1½ miles east from Zion Quarry, did not prove a success, as stated previously, owing to large clay seams which appeared throughout the rock after shooting. This quarry was worked, however, from March 17th, 1913, when stripping was started, until August 31st, 1913, the same methods of drilling and shooting being used as at Zion Quarry. A locomotive was assigned to its service, as at Zion, and two steel guy derricks were worked. The crusher muck was hauled to the crusher at Zion and unloaded there in the same manner as at Zion Quarry. The stripping at Ellison was done by teams and a steam shovel, there being a heavy overburden against the toe of the hill.

Storage.—Whenever a surplus of crushed stone accumulated in the bins at Zion, it was drawn out into 7-yd. dump cars and stored in a pile alongside a 10-ft. embankment. About 8 000 cu. yd. were stored and afterward picked up by a locomotive crane, operating a ½-yd. clam-shell bucket. By storing materials, the crushing and quarrying operations were carried on continuously, and a shutdown at the dam did not affect the quarries.

Zion Quarry was closed on December 24th, 1913, all the camps were removed, and the plant was shipped out.

Concrete.—As many different classes of material were used in the work, the coarse-aggregate bin, having a capacity of 800 cu. yd., was roughly divided into two sections; all crushed stone was dumped in one end of the bin and all washed gravel and Elmore gravel, consisting of 55% gravel and 45% sand, were dumped in the other end. The sand section of the bin was separated from the coarse-aggregate section by a partition. The crushed stone and gravel were drawn to the conveyor belt at the same time, in about equal proportions, and were elevated into the coarse-aggregate hoppers above the mixers. The sand was drawn by itself and ran into a separate bin, also above the mixers. There was a charging hopper just above the mixers, and all materials were drawn from the bins above into this hopper, where they were properly proportioned.

The mixing of the concrete was watched by an inspector on duty at all times at the mixer plant, and he was constantly in touch with

the chief inspector on the dam, who also with his assistants kept close watch on the concrete coming to the dam.

Three mixtures were used: 1:3:6 in the main body of the dam and the heavy sections of the power-house; 1 : 2½ : 5 in the cut-off trench and bottom 2 ft. of the foundation of the dam; and 1:2:4 around the scroll casings, penstocks, and all reinforcing in the power-house. The greater percentage of crushed stone came from Zion Quarry, the remainder being brought from outside quarries and shipped to the dam by rail. In addition to the crushed stone, 31 034 cu. yd. of gravel were used, principally around the complicated curved forms in the power-house and in filling the stream-control openings.

In general, as shown by the analyses of a great many different mixtures used in the work, there was about 10 to 15% too much fine aggregate used. The writers, in view of the results obtained, however, do not consider this a bad feature. There were no honey-combed walls in either the dam or the power-house, and only a few seepage spots appeared in any places except at the vertical expansion joints, the showing in this respect being excellent. When broken off in large chunks, the concrete was dense and had a uniform texture; and when the forms were removed, the outside appearance of all walls was good, no finishing being necessary except in spots where pieces had been broken out by bolts, etc.

Expansion Joints.—At first, expansion joints were placed 108 ft. apart, and after two of them had been started, the distance was cut down to 72 ft. The 108-ft. block proved a little too large to complete satisfactorily as the concrete at one end set before it did at the other. All joints were painted with a heavy coat of hot Barrett pitch. These joints were very effective, as far as could be noted at the end of one winter, and no cracks of any kind developed in the dam.

Handling and Placing the Concrete.—The concrete was dumped from the mixers into 2½-yd. buckets on small cars, or directly into chutes. The cars were hauled by mules to the cableways, two cars being used for each mixer. There were four tracks under the cableways, drawing closely to each other in pairs at the mixers. See Figs. 22 and 40. The buckets were picked up by the cableways and placed at the points needed. No. 1 mixer also dumped directly into a chute. Chutes were used wherever possible to deliver concrete to buckets traveling on small cars operated with a hoisting engine. A large

portion of the power-house concrete was passed through chutes, and the system when properly laid out is most effective. The chutes delivered the concrete to buckets on cars which were taken to the dam, *via* the Low-Level Line and the up-stream coffer, by a dinky locomotive. This method was as effective and rapid as any used on the job. A great deal of the concrete was dumped directly from the cableways, and this method was fairly rapid. However, the maximum 10-hour run with cableways was seldom more than 400 cu. yd. The writers believe that cars and derricks, where it is possible to use them, are more efficient and rapid than cableways. Where narrow forms are to be concreted, the cableway is exceedingly slow and should not be used if any other method can be found.

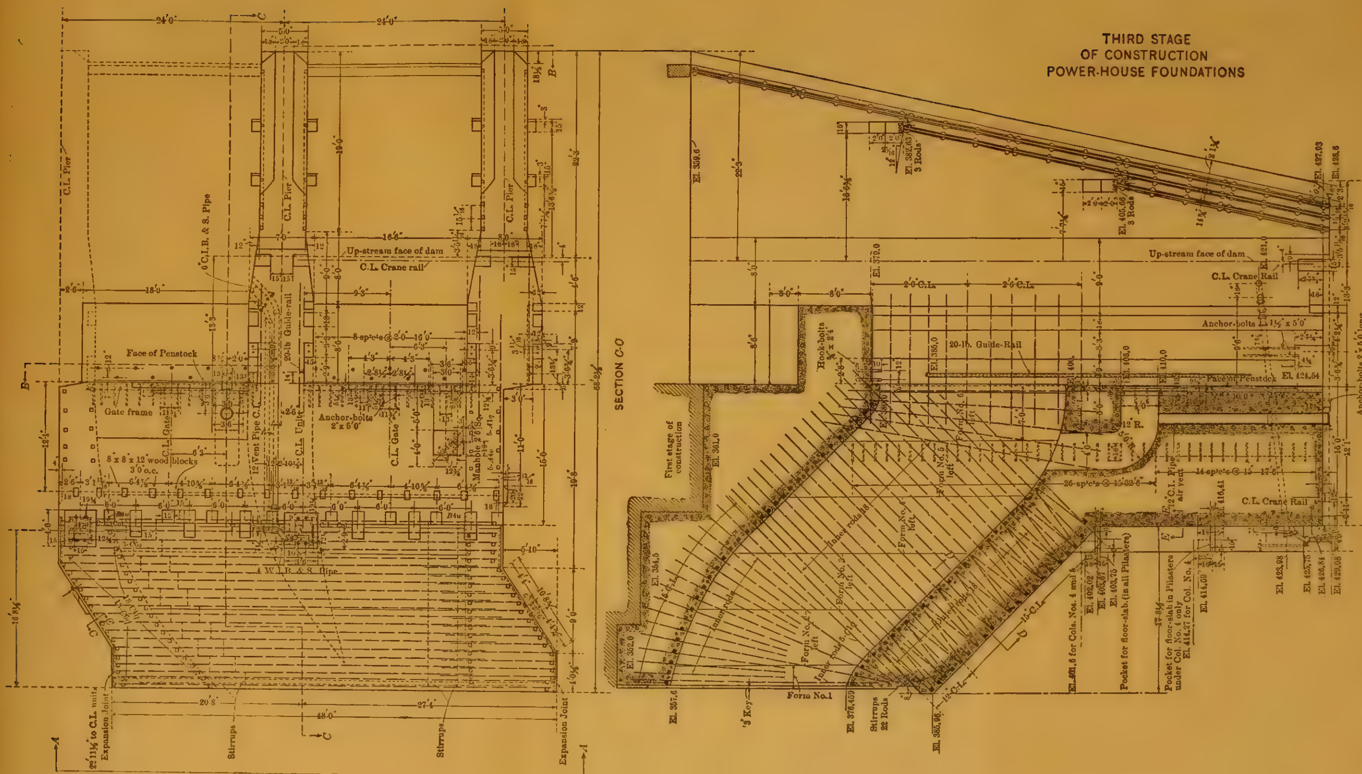
Cyclopean stones were delivered directly to the derricks on the coffer-dam, and, between buckets of concrete, were set by the derricks. Each stone was carefully washed and cleaned, and, after it was placed, it was carefully shaken with a bar and bedded. In the dam the percentage of cyclopean stone was small, for the reasons mentioned in discussing the quarry operations. The horizontal joints between pours were always left bedded with as many cyclopean stones as possible, in order to get a thorough bond with the next layer.

A total of 6 482 cu. yd. of cyclopean stone was placed in the dam, or 5.1 per cent. In the power-house foundations there were 1 188 cu. yd.

There were two types of chutes on the work. The first was made of No. 12 gauge metal and was 10 in. in diameter. It was of the semicircular type, and was served by a special bottom-dumping car at the mixers. The mixer dumped into the car, and the car was pushed by hand over a hopper to the chute, and dumped into it. This chute was not a success, the main objection being that it was too small. It clogged frequently, and took too many men strung out along it to operate it and keep it clear. The metal was too light and wore out too quickly. This size and type are not recommended for similar work. This chute was abandoned, and a home-made one was built, which gave satisfaction. It consisted of a trough, built in 6 to 8-ft. lengths, with sides about 18-in. high, and well braced; $\frac{1}{8}$-in. sheet iron was rolled in 8-ft. lengths to a half circle, 14 in. in diameter, and fastened into the bottom of the trough, giving a chute with high sides and a rounded bottom. It was of sufficient size, discharged on slopes of 18° rapidly and without clogging, provided the concrete was wet

PLATE XXVII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1914.
SAYERS AND POLK ON
HYDRO-ELECTRIC POWER DEVELOPMENT
IN ALABAMA.

THIRD STAGE
OF CONSTRUCTION
POWER-HOUSE FOUNDATIONS



enough to flow, cost about two-thirds of the price of the first chute, and its life was nearly twice as long. It is to be noted that concrete discharged down a 75 to 100-ft. chute was as good as that which fell only a few feet from the mixer to the bucket, and on dumping the two side by side in the forms no difference could be discerned.

After leaving the mixer platform, the concrete was delivered to a derrick, *via* some one of the routes mentioned, and by it placed in the form. An inspector in the form saw that the concrete was brought up uniformly throughout, no racking being allowed. When it was seen that the form could not be completed in one pour, a temporary saw-tooth bulkhead was put in to prevent thin-edged layers. The inspector saw that all concrete was well worked after being dumped; when it lacked sand, it was worked with shovels and tamped until the mortar had worked its way all through it. All faces against forms were carefully spaded. Two bars were used in bedding the cyclopean stone in the soft concrete.

Before making a new pour on the surfaces of old concrete, they were thoroughly cleaned in the following manner: Two gangs of from 12 to 14 men with foremen, when the work was at its height, were kept cleaning forms only. Whenever possible, the surfaces of the old concrete were cleaned about 12 hours after pouring, while they were still green, all scum mortar and "laitance" remaining on top were carefully picked or shoveled off, and the surface was washed until it was absolutely bright and clean. Great stress was put on this particular feature. Concrete was not allowed to be placed in any form until it was cleaned to the satisfaction of the engineer. After the work got under way, and the amount of cleaning insisted on was fully comprehended by the contractor's superintendent, there was little trouble. Also, after a form had been properly washed and cleaned, a 1:2 mortar was dumped in and swept over the surface with a wire broom just before the concrete was dumped. Leaks through horizontal joints have not appeared in any portion of the dam or power-house, and the writers attribute this to the care exercised in cleaning and working the surfaces of old concrete as described.

Power-House Concrete.—In general, the power-house concrete was poured in the same manner as that for the dam, except that the mixtures varied, as noted elsewhere. Fig. 41 is a general view of the power-house operations on July 2d. The heavy blocks indicated in

the first stage of construction, Fig. 43, were of cyclopean masonry, the remainder being mass masonry. The power-house concreting was prosecuted in four separate stages, and complete details of each of these stages are given on Figs. 43 and 44, and Plates XXVII and XXVIII, respectively. These drawings were followed closely in the field, and were of great assistance. The general plan to be followed was carefully discussed at the beginning of the work, and a line of action definitely decided. Fig. 42 shows the second stage of concreting completed. Fig. 45 shows the second stage completed and the forms for the third stage, that is, the concrete around the penstocks, being placed. In Fig. 46 these forms are shown erected. Fig. 47 shows the down-stream elevation of the power-house foundations, the photograph having been taken from the tail-race. Fig. 48 is a close view of the interior of the draft-tube; the man standing in the center of the opening giving a fair idea of its size at the exit.

General.—On April 12th, the first concrete was poured in Section 1 of the dam, immediately next to the wasteway section. Construction of the wasteway section followed immediately, and then the remainder of the dam in Coffer-dam No. 1, and the heavy foundation blocks in the power-house, shown on Fig. 43 as Stage No. 1.

On May 13th, a night shift on concrete was organized at the dam. On May 26th, concreting in the last block of the dam inside Coffer No. 1 was started, and Fig. 31 shows the general condition at this time. On May 31st concreting was started in Section 6 of the power-house. On June 4th, No. 2 mixer was started and the concreting was begun on the bottom cradles for Draft-Tubes Nos. 5 and 6. On June 13th, the filling of the large natural hole in the foundations under Units 1 and 2 was started. By July 12th the concreting and other work had progressed to such a stage that the water in the river was turned through the tunnels left in Section 1 of the dam. On July 25th, Draft-Tube No. 1 was concreted to grade and ready for the scroll casing. On August 26th, concreting was begun in Coffer No. 2, next to the east abutment, which latter lagged behind on account of a bad mud seam which had to be worked out, and on which concreting was not started until September 22d. On September 5th, the first section of rollway in Coffer No. 1 was reached and poured, this marking the completion to Elevation 406 of a portion of the first section of the dam. On September 27th all the rollway section, inside Coffer

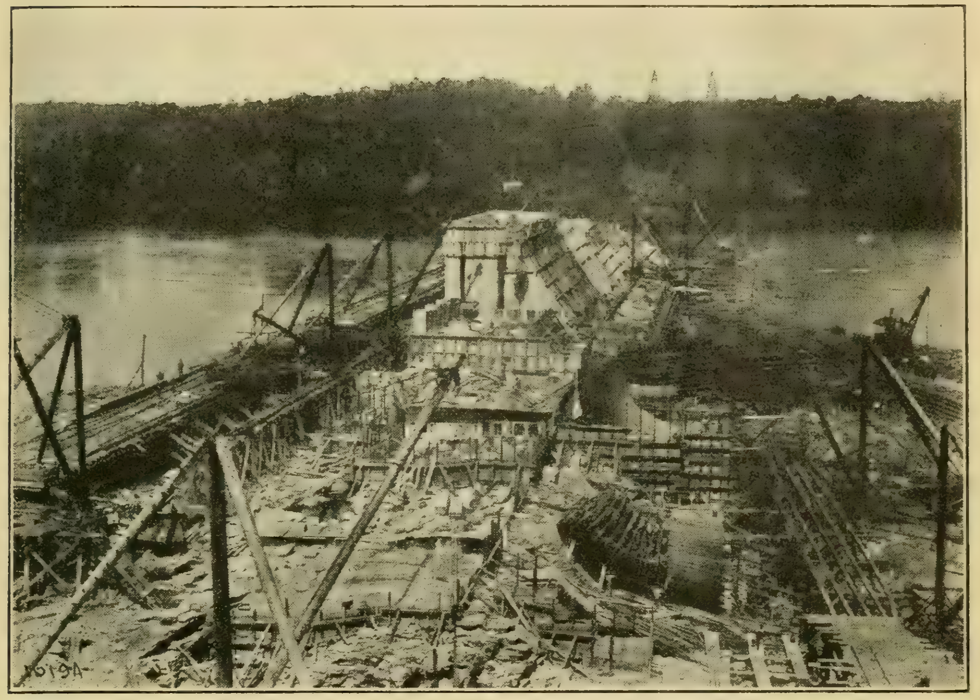


FIG. 41.—CONDITION OF WORK, JULY 2D, 1913. FIRST STAGE OF CONSTRUCTION OF POWER-HOUSE STARTED.

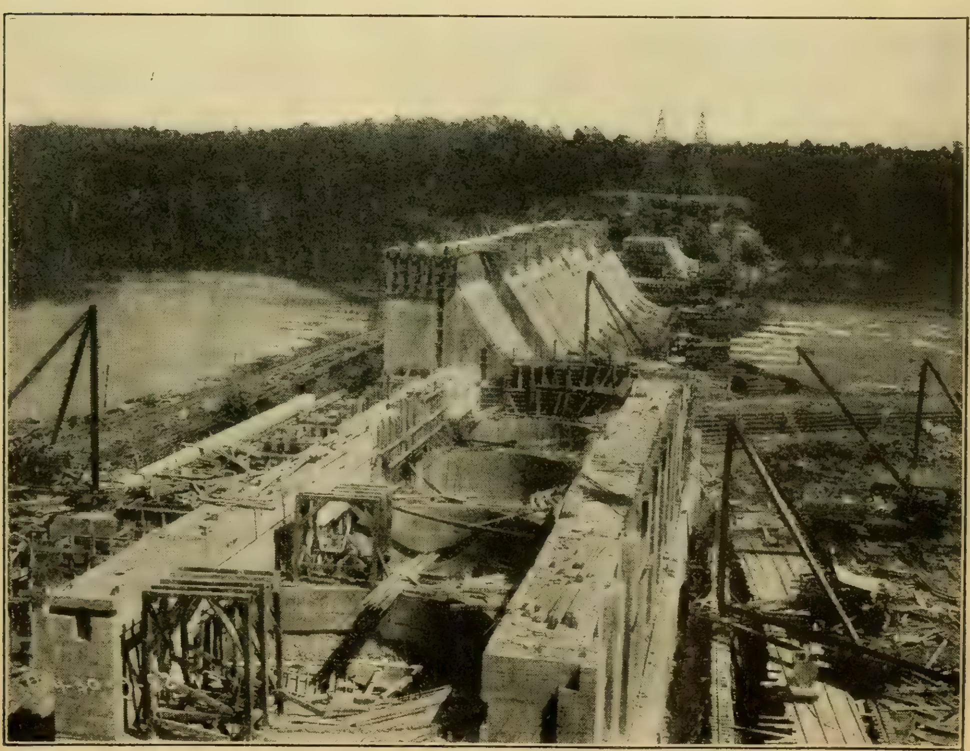


FIG. 42.—GENERAL VIEW OF WORK, SEPT. 22D, 1913. SECOND STAGE OF CONSTRUCTION PRACTICALLY COMPLETED; CONCRETING UNDER WAY.

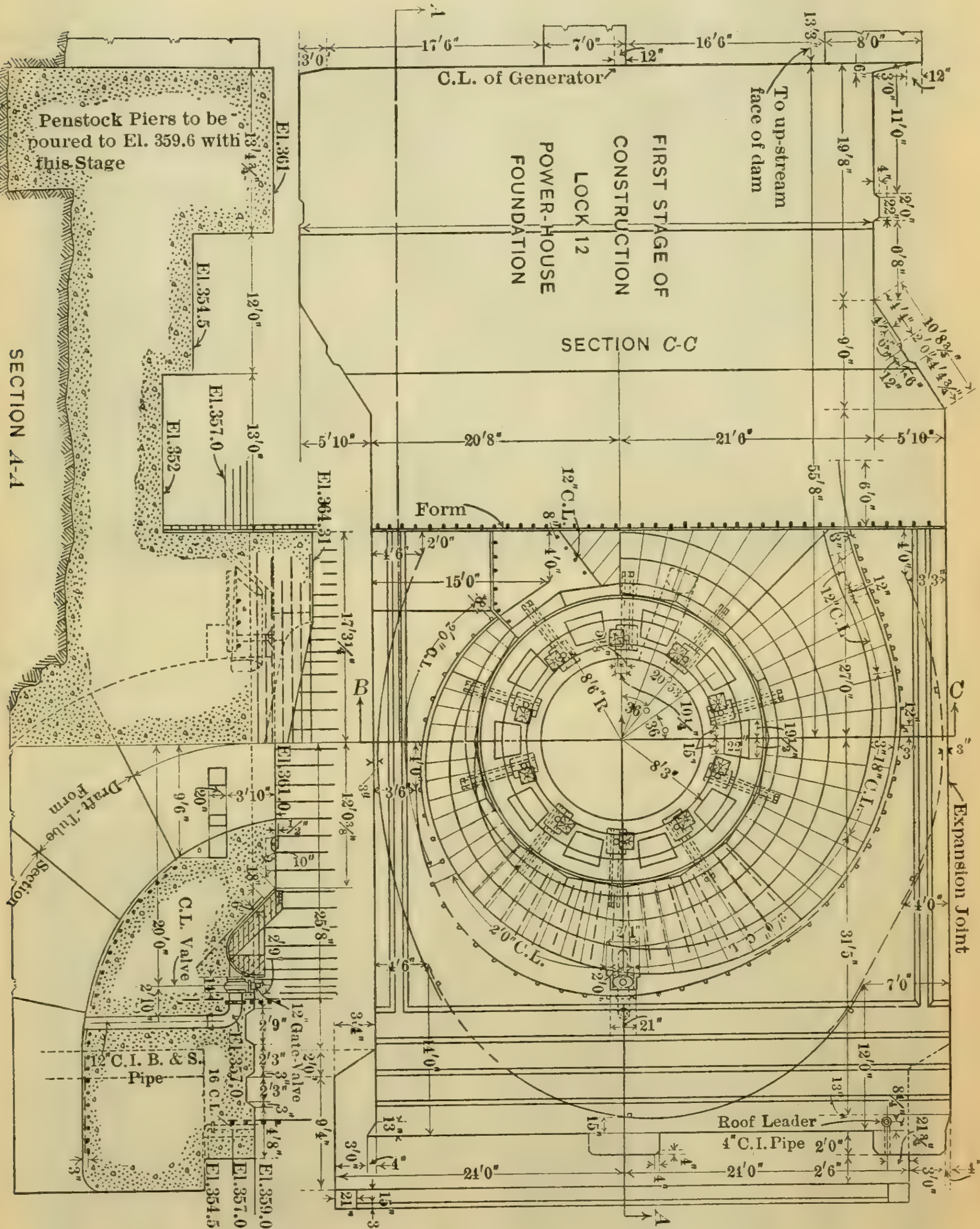

Penstock Piers to be poured to El. 359.6 with this Stage
C.L. of Generator
To up-stream face of dam
FIRST STAGE OF CONSTRUCTION
LOCK 12
POWER-HOUSE FOUNDATION
SECTION C-C
SECTION A-A
Form
Draft Tube Form
Section
C.L. Valve
12" C.I. B. & S. Pipe
12" Gate-Valve
Roof Leader
4" C.I. Pipe
Expansion Joint
El. 361
El. 354.5
El. 357.0
El. 352
El. 364.31
El. 361.0
El. 359.0

FIG. 43.

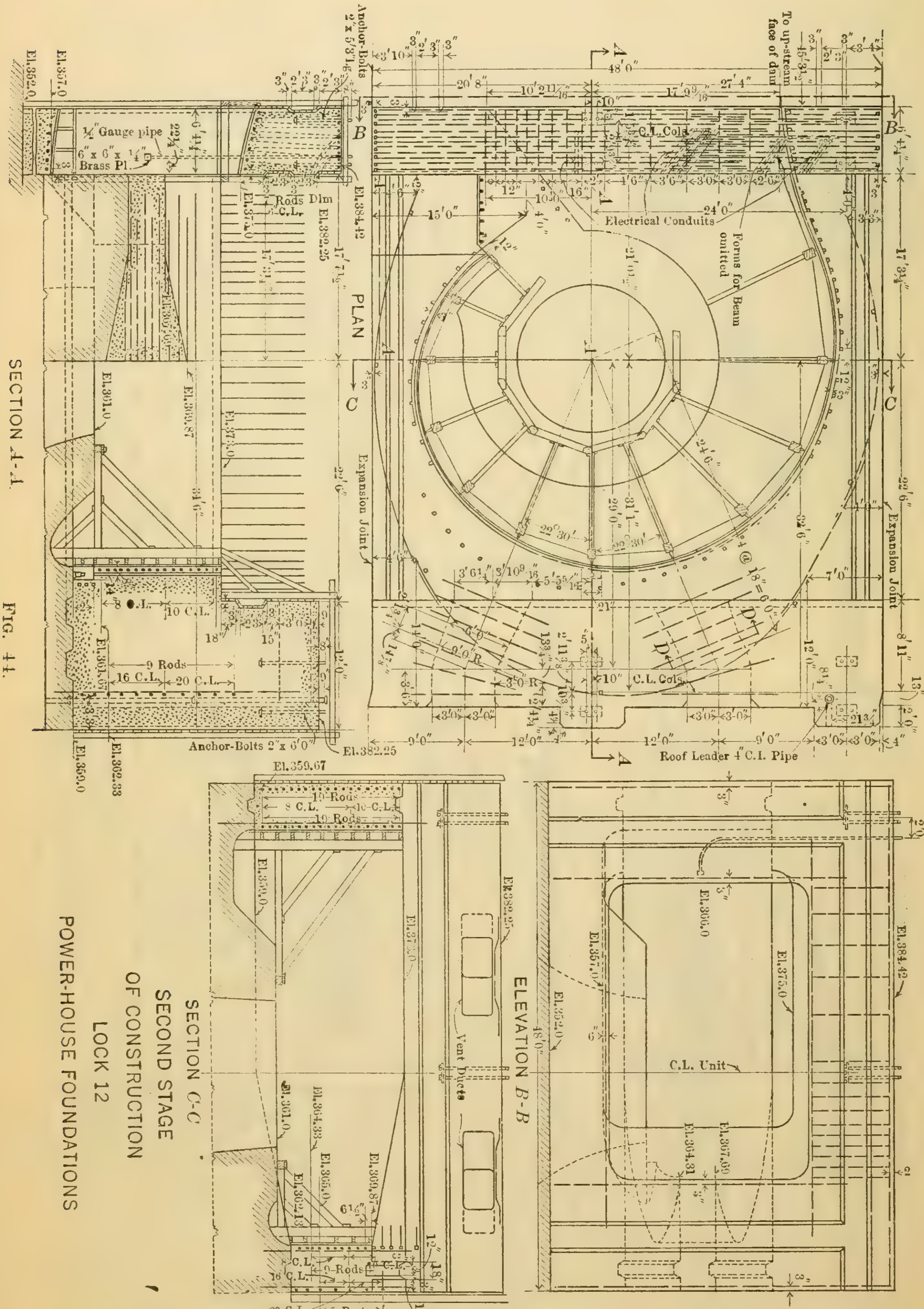


FIG. 44.

SECTION C-C
SECOND STAGE
OF CONSTRUCTION
LOCK 12
POWER-HOUSE FOUNDATIONS

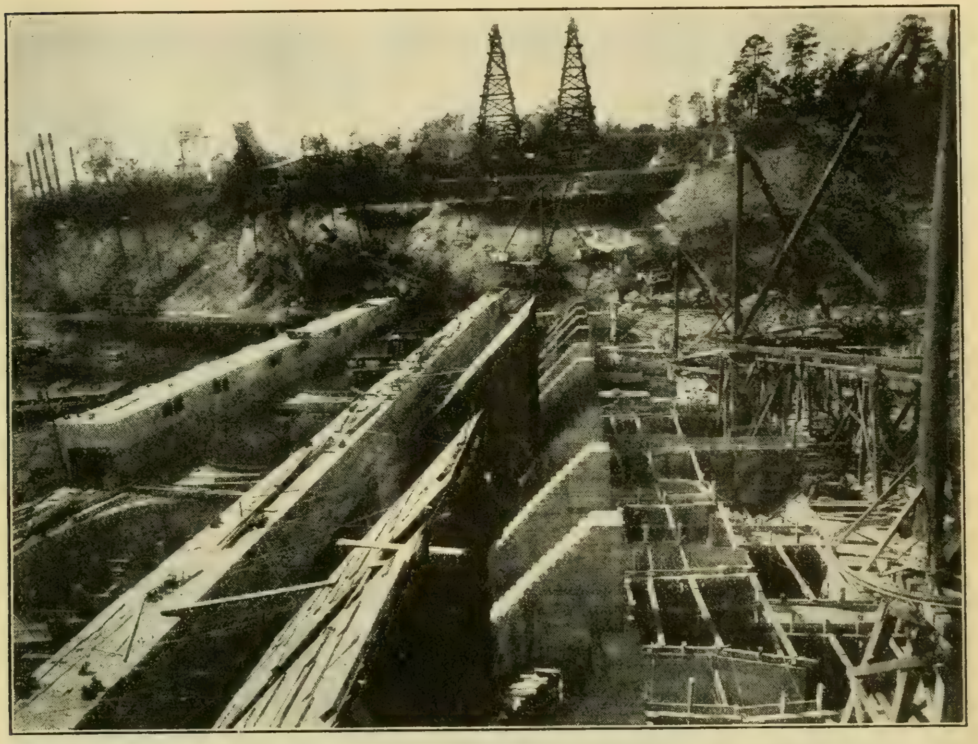


Fig. 45.—Cradles for Penstock Forms. Second Stage of Construction Completed.

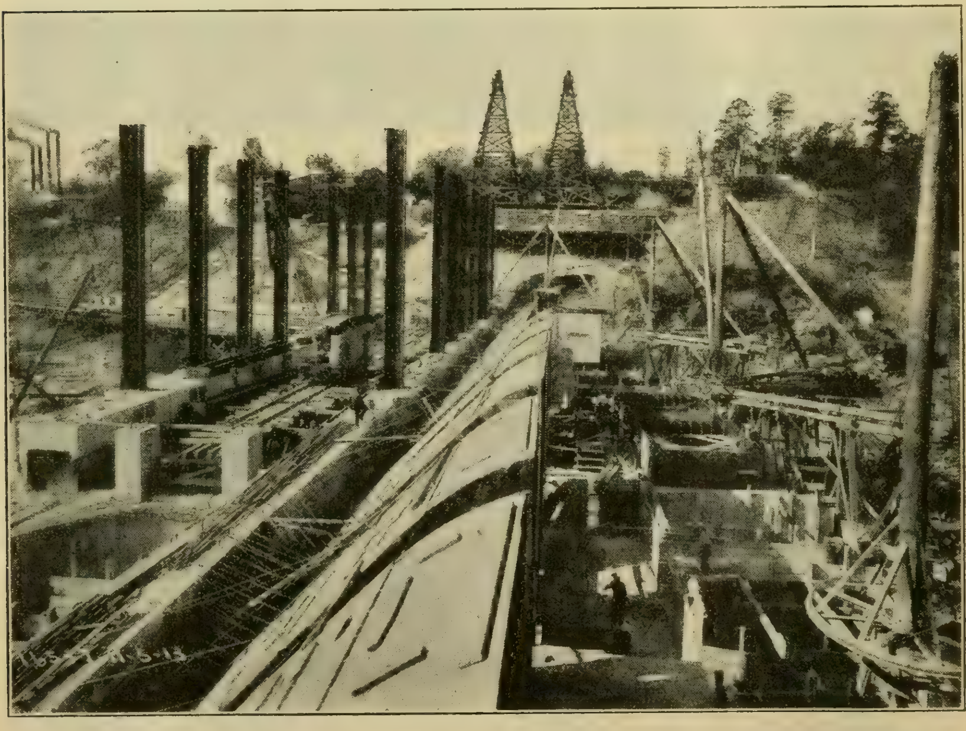


Fig. 46.—Penstock Forms under Construction. Locomotive Crane Setting Up Columns of Power-House.

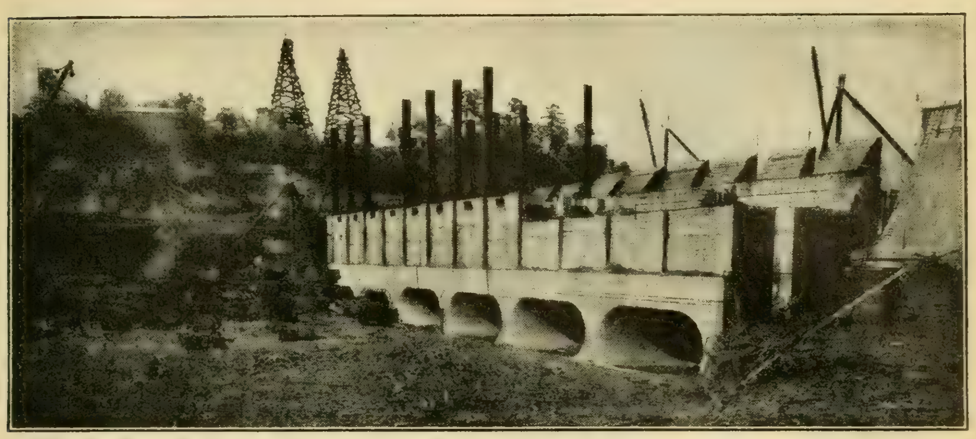


FIG. 47.—DOWN-STREAM VIEW OF POWER-HOUSE FOUNDATIONS, AFTER COMPLETION OF SECOND STAGE OF CONSTRUCTION.

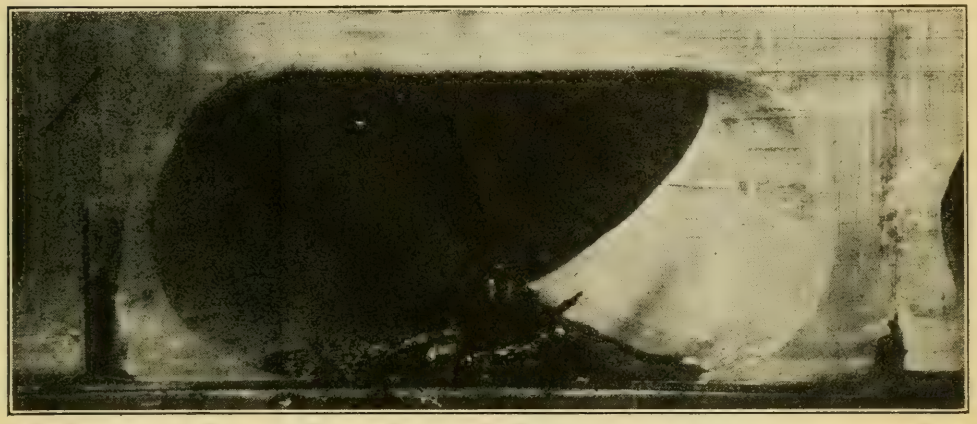
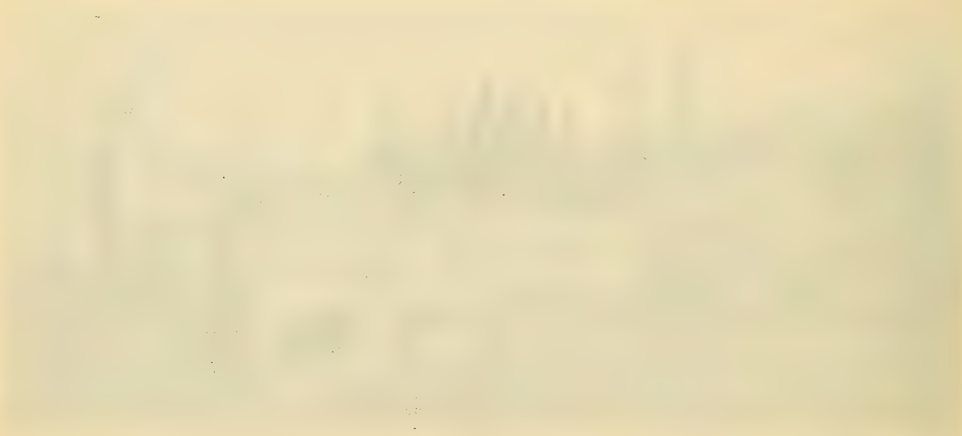
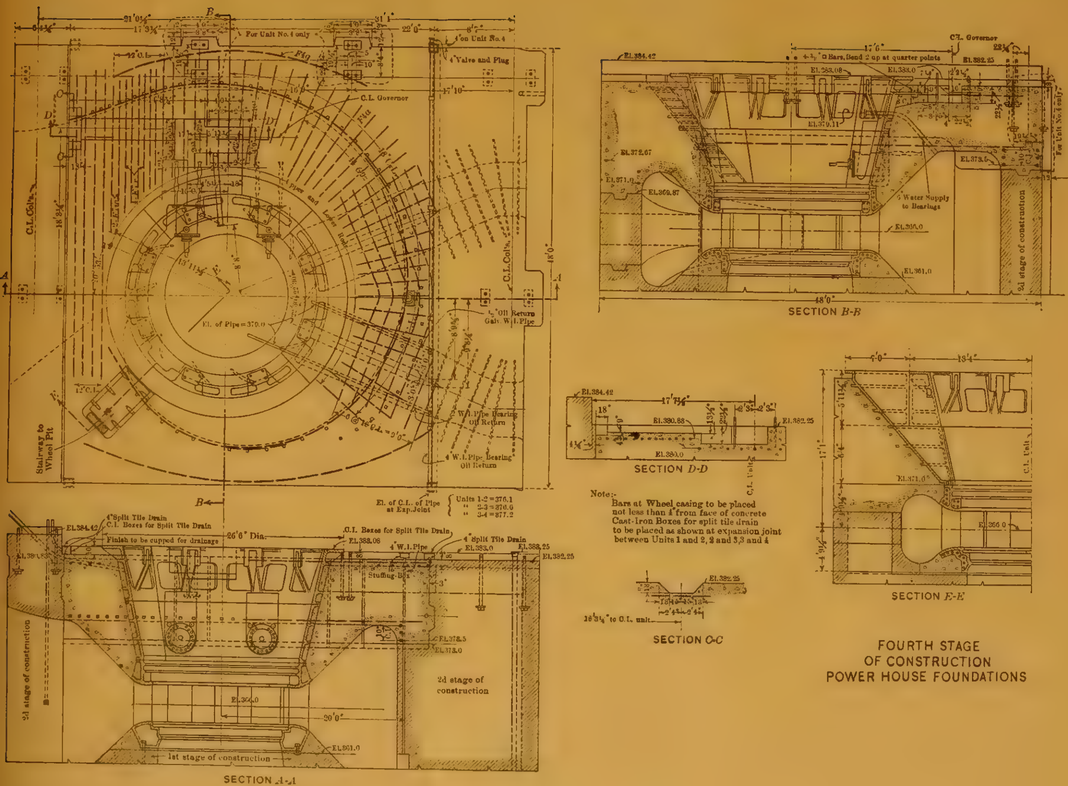


FIG. 48.—LOOKING INTO DOWN-STREAM END OF DRAFT-TUBE.

FIG. 49.—CREST OF SPILLWAY BEFORE CONSTRUCTING PIERS.





FOURTH STAGE
OF CONSTRUCTION
POWER HOUSE FOUNDATIONS

No. 1 (Fig. 49) reached Elevation 406.0, the crest, and the concrete all along the front of the power-house had reached Elevation 380. On October 13th, the highest run on the concrete was made, being 2 220 cu. yd. for two 10-hour shifts, the day shift alone putting in 1 120 cu. yd. in 10 hours. On October 18th, concreting was started around the first penstock forms, and on October 21st pouring was started on the last section of the dam (No. 9). On November 8th, the concreting of the bottom 4 ft. of the eleven 15-ft. stream-control openings was started. It was completed on November 21st. On November 25th, the dam for the entire length of the spillway was completed to Elevation 406, the crest. On December 20th the power-house concrete reached an elevation of 416 in front, and all the scroll casings, draft-tubes, etc., were ready for the setting of the speed rings. On January 3d, the filling of the stream-control openings was begun, and the work was completed on January 31st. On February 15th, the last wheel casing was set and the last scroll casing poured, completing the operating floor of the power-house to rough grade. On February 28th, the last concrete in the spillway piers was poured, completing the work, and on March 6th, the work of tearing out the mixers was started. From the day that concreting started to the last yard poured, 10 months and 18 days elapsed, and 187 802 cu. yd. of masonry were placed. Fig. 49 shows the finished rollway section before the piers were constructed.

Spillway Piers.—The spillway piers above Elevation 425 were so small in plan and contained so much iron framework that it was very dangerous to dump concrete into them from the cableways. A small derrick was rigged up on a standard flat car—the hoist being operated by air, as usual—for the pouring of these small piers in three 6-ft. lifts each. The cableway delivered concrete to this derrick car. Some 18-in. I-beams, which were bought for the purpose of carrying the traveling gate-operating mechanism between the piers, were used temporarily to span the back of the main piers at Elevation 425, and carry a standard-gauge track for the derrick car. These piers are shown in plan and section on Fig. 19.

Discussion of Methods Used.—In general, as previously stated, the writers do not consider cableways as the most satisfactory method of handling concrete, in view of the experience gained on this job. The system of belt conveyors and elevators for concrete materials is one to be avoided, if possible and if space permits, as a fruitful source

of delay. There was one delay of a week on this work, due to a broken elevator belt, and it was expensive. A gravity plant should be built, if it is possible to lay it out. Handling concrete by buckets on flat cars, with dinky locomotives to the derricks, is preferable to the cableway.

The plant, as laid out and used, was effective and, in 1 month, more than 35 000 cu. yd. of concrete were handled with it, but the writers believe that a gravity plant would have been an improvement and more effective. The point which it is desired to illustrate is that a gravity plant should be carefully sought first, and that cableways should not be made the main method of concrete transportation.

Reinforcement in the Power-House.—Complete and detailed drawings were furnished for the correct cutting, placing, and setting of all reinforcement. All curved rods were laid out on a large platform near the tail-race below the power-house, and there bent with a bender to the curves shown on the drawings. All stirrups, etc., were also made there.

Spacing rods were used, to which the regular reinforcing rods were wired. Large nails were driven into the forms, and the rods were wired to them in order to keep them at the proper distance from the forms. In some cases suspended frames were used to hold the rods in exact position until they were concreted. In all cases particular care was taken to keep the rods the proper distance from the forms and properly spaced. All concrete poured around the reinforcement was mixed in the proportion of 1 : 2 : 4.

Completing the Work.

Final Closing.—When all concreting in the dam had been finished, except the stream-control openings, and all piers had been concreted to Elevation 417 on the spillway, and the concreting on the power-house and west abutment to Elevation 420, it was deemed safe to close the gates. Fig. 16 shows these gates hung and ready to close. Previous to the closing, when it was seen that the work was reasonably safe, the west eleven 15-ft. openings in Section 1 of the dam were blocked off by placing wooden gates about 7 ft. high in front of them, one at a time, and a concrete wall, 4 ft. high and 6 ft. thick, was placed next to the bulkhead. This small wall was built first to control the leakage that came through these 7-ft. bulkheads, the water coming

through being collected in an interior box sump in the concrete and led through the wall down stream in pipes which were afterward grouted under pressure. The main part of the opening was then filled to the same height throughout as this front wall, no trouble from leakage being experienced. Each of these eleven openings was treated in the manner described, except that on the five westerly openings, which were built primarily for a stop-log gate, the permanent stop-logs intended for them were dropped into the bottom 7 ft. of the guides and left in. The bulkheads in the remaining six openings were removed after the concrete had set, in order to give a smooth clean sill for the flap-gate to butt against. This work was started on November 8th, 1913, finished on November 24th, 1913, and served two good purposes. It was done primarily as a precautionary measure; the bottom of these tunnels was very close to the natural bed of the river, from 12 to 18-in. on most of them, and with such a small sill for gates of the flap type to strike against, there was a chance for a stone or sunken timber to lodge against it at the time of closing and give a lot of trouble by preventing the gates from seating properly. With this additional 4 ft., the bottom of each tunnel was brought well above the bottom of the river, and also well above the tail-water elevation. There were two gates to each opening, and of the eleven openings, five were of the stop-log type, shown by Fig. 15, and six were of the flap-gate type, Fig. 16. When all these eleven openings had been filled with from 4 to 6 ft. of concrete as described, the ten smaller 8 by 12-ft. gates, which were of the flap type, were hung and securely closed on December 21st, 1913, and caulked around the outside. No water was running through these openings at the time, their bottoms being only 4 ft. below the tops of the large openings. The closing of these gates was a very simple and easy matter, and they were caulked and inspected from the upper side. On December 21st also, five of the larger openings were closed with the ten stop-log gates. The closing of these gates was easily accomplished and did not raise the water very much, as only a small quantity was running through this particular set, the water stage being very low. These gates were then securely wedged and caulked on the outside, up-stream face, and the filling of the tunnels was started. Next, a light template of the larger type of flap-gates still to be closed was made, and each of the openings was carefully checked with this template to be sure that there were absolutely no obstructions to the closing of the gates

and that nothing of any kind would foul them while they were being lowered. This check was made several times and just previous to the closing.

Owing to the non-delivery of the penstock steel gates by the manufacturers, it was deemed necessary at this time to drop stop-log gates in front of each penstock opening, there being six penstocks with two gates each. As the time was very short and the weather was threatening, it was not possible to secure timbers large enough to withstand a maximum head, consequently, the logs were dropped into the gate grooves and reinforced by four 18-in. I-beams. Later, logs of the proper size were dropped into the grooves regularly provided, the I-beams and logs were removed, and the steel gates put in. The I-beams were those bought for carrying the spillway operating mechanism from pier to pier. All these stop-logs were in and properly bolted and caulked on December 27th. On December 28th, at 7 A. M., the large flap-gates remaining were lowered to a horizontal position, clearing the water by about 2 ft. and rigged up so that by attaching a loose line on one side of the cables holding it above and cutting another loop the gate would fall, but the cable would be held by the loose line and not catch under the gate as it swung shut. On December 27th, rain commenced, and the river, which had been rising gradually all the while, started to rise rapidly, and it was decided to close the remaining gates at once, everything else being in readiness. The gates were closed in the order and at the times shown in the following:

Gate No.	Time closed.
*12	11 : 20 A. M.
11	1 : 10 P. M.
10	1 : 13 "
9	1 : 18 "
8	1 : 22 "
7	1 : 26 "
6	1 : 28 "
5	1 : 47 "
4	1 : 51 "
3	1 : 54 "
2	1 : 57 "
1	2 : 00 "

* Gate No. 12 was first lowered in order to try out the method. The closing of the dam really started with Gate No. 11.

Each gate, after being cut loose, followed three distinct motions: First, a rapid fall of 2 ft. until it struck the water; second, a very slow swinging in with the current until within 3 ft. of closing; and third, a quick, abrupt slam against the concrete face of the wall. The leakage was very small after closing, and was taken care of as described later.

Before the final closure was made, all the gates in the wasteway section were examined carefully, and raised and lowered to test them out in the dry. Plate XXVI shows this wasteway section. Two sets of these gates were left open, and water passed through them during the period of filling the reservoir and for some time afterward, serving to hold the water on the crest of the spillway a little lower while the tunnels were being finally concreted. The water in the reservoir rose with fair rapidity, due to heavy rains, and flowed over the crest of the dam at 9 A. M. of January 1st, 1914. During the period of filling, a careful watch was kept of the river bed below the dam and in the vicinity of the dam itself for any indications of leaks or springs. None whatever was seen, and the dam and rock were absolutely tight.

Concreting the Openings.—In order finally to fill the openings through the dam, the water pouring over the crest had to be diverted and kept out while the work was being done. This was accomplished by building six gates, 3 ft. high and 30 ft. long, reinforced on the back with hog rods, and dropping them into the gate slots provided for the spillway gates. There was only a depth of about 18 in. of water over the crest at this time. All leakage under these gates was stopped by dumping clay and cinder just outside of them, their seats being about 3 ft. from the front face of the dam. It was then necessary to take care of leakage through the openings below and devise a scheme for filling them effectually. The leakage was taken care of in the following manner: A water-tight bulkhead was built about 8 in. back from the inside face of the final closure gate, and a sufficient number of drainage pipes was run from this bulkhead to the downstream face of the dam to take care of all the water, the leakage being taken care of in most openings by from two to four 3-in. pipes. These pipes were afterward filled with cement grout under a pressure of from 10 to 40 lb. per sq. in. In order to pour the concrete into the openings directly from the cableway, hoppers 6 ft. square were built around the top shaft of each tunnel, but only one shaft was used

for filling purposes, the second shaft, being of no practical value, was not used. The concrete was dumped directly from $2\frac{1}{2}$-cu. yd. buckets into these hoppers and discharged down the shafts into the tunnels, where it was handled by men with shovels to within from 3 to 6 ft. of the roof, when work was stopped and the concrete allowed to set for at least a day. In the first few openings filled, it was thought best to build a concrete wall about 6 ft. thick of a medium dry concrete, to tamp it thoroughly, and to get a tight positive seal. As the wall did not prove to be water-tight, the method was discontinued. The method finally adopted for securing an absolutely tight seal was, as already partly described, to fill the tunnel throughout its whole length to within from 3 to 6 ft. of the roof, the men carefully spading and working this concrete with shovels so as to make a tight job. After this had set for 24 hours, it was carefully washed and cleaned and then a good flowing gravel concrete was dumped into the shaft, vent pipes having been properly placed, which vents also acted as tell-tales as the filling progressed. The back form was bolted up tight to the curve of the dam, and it was found that the dumping of this concrete down the main shaft would force concrete up and out of the back shaft to a height of 35 ft. or more above the top of the tunnel. The writers are of the opinion that, with a pressure of this kind and with a free-flowing concrete, every corner of the tunnel was bound to be tight, especially after the above described demonstration. As a further safeguard, however, a system of grout pipes was put in, and afterward grout was forced into these to fill any possible cracks.

In the ten small openings on the east side of the river, the same general scheme of filling was followed out, that is, a water-tight bulkhead was built 8 in. from the gate, leakage was carried off by pipes to the down-stream side of the dam, and concrete was poured to within 3 ft. of the top and allowed to set hard. Two 2-in. grout pipes were used, one on each side of the tunnel, and as close to the top as practicable, but below each keyway a **T** had been placed in the pipes and a nipple used so that its end would get within $\frac{1}{2}$ in. of the top of the keyway. Very little grout was placed in these pipes; in fact, concrete in nearly all cases completely filled the tunnels.

One difficulty which should be guarded against in similar operations, was experienced in filling these tunnels: If the sand used was very coarse, as it was in one or two instances on this work, the concrete

would fail to run, and the tunnel would block itself. The remedy adopted by the writers in this emergency was immediately to dump 2-yd. batches of fairly rich mortar until the whole mass started moving again. As a rule, this remedy was satisfactory, and as soon as the mass started moving it came with a rush. In the worst case encountered, twenty-two 2-yd. batches of mortar were used to seal the top effectually. After grouting and setting a proper length of time, the openings were all tight, and showed very small negligible leaks which shortly clogged themselves.

Section D—General Organization of the Contractor and Power Company on the Work.

General Organization of Contractor's Forces.—The main features of the contractor's organization were as follows: A General Superintendent, with headquarters at the dam, was directly responsible to the New York office of MacArthur Brothers Company. He was also responsible for the execution of the contract, and worked directly with the Power Company's Resident Engineer on the job.

His assistants were: Superintendent of the Dam, Superintendent of Railroad, Superintendent of Quarries, Office Manager, and Master Mechanic. All other parts of the organization were handled under these general departments, each department being complete within itself, yet working closely with all the others.

The Superintendent of the Dam had charge of all day and night forces at work at the dam, except such as came under the Office Manager and Master Mechanic. He had a Night Superintendent, and, on both shifts, walking bosses and foremen in all the various departments of the work.

The Master Mechanic at the dam looked after the proper maintenance and operation of all machinery and plant, and all derrick runners, engineers, firemen, machinists, helpers, blacksmiths, etc., were under him. This department worked in very close relation with the Superintendent of the Dam, and derrick runners, cableway runners, pump men, etc., worked to a large extent under both departments. This relation between the Superintendent of the Dam and the Master Mechanic is not in general a wise one; in order to get the best results, the Master Mechanic should be a subordinate of the Superintendent of the Dam; however, in the case in hand, the scheme

worked satisfactorily. The machine shop was about $\frac{1}{8}$ mile from the dam, and a large part of the Master Mechanic's time was spent there.

The Superintendent of the Railroad was in complete charge of all railroad operations outside of the Lock 12 and quarry yards. He had a force of trainmen, yardmen, trackmen, dispatchers, etc., under him, and was held responsible for the movement of all material over the railroad. His headquarters were at the junction of the main line and the quarry spur, 7 miles from the dam, known as Camp No. 4.

The Superintendent of the Quarry had charge of all quarrying and crushing operations at Zion and Ellison Quarries, and the camps at these two places. He had day and night forces, with walking bosses, foremen, a master mechanic, timekeeping and clerical force, commissary force, and all the skilled mechanics, engineers, etc., necessary for the work.

The Office Manager handled all commissary matters, purchases of supplies, timekeeping, bookkeeping and all financial matters. He had at Lock 12 a general office force of bookkeepers, timekeepers, stenographers, clerks, etc. All commissary managers at the various camps came under his jurisdiction, as well as timekeepers and material checkers there.

Commissary Department: Organization and Methods.—The Commissary Department deserves special mention, as it was a most important part of the work. Supplies for all four camps and for about 2 000 men were handled through this Department, all kitchens, cooks, as well as stores, coming directly or indirectly under it. The store at Lock 12 was not very well situated, and should have been larger. The writers believe that, had a little more thought been spent on its planning, it would have brought in better returns than it did. No elaborate stock was carried, and no particular attempt was made to realize profits from merchandising. The writers feel confident that it would pay the expenses of all commissary forces to run a store of this kind along proper lines. A store operated just outside of the limits carried a stock of merchandise and fancy groceries, and, even though big prices were charged, a good profit was made by the owner. The writers would recommend that, on a job of this size, the commissary store be a large airy building, centrally located, have regular counters, shelves, and tables for the display of goods; it should have a butcher shop as one department, and also a cold drink stand. It is to be

noted that the small cold drink side line operated in connection with the commissary practically paid the wages of two clerks alone in the summer, and one gave only about half his time to it. One general commissary inspector should be appointed, and he should be experienced in operating a store of this kind. All the various camp commissaries should be under him, and it should be his duty to inspect and watch constantly the character and class of goods demanded and to fill that demand.

It is astonishing how negro laborers and others on construction work will spend all they make for good clothes, candy, fancy groceries, etc., and a good profit can be made from these. It should be the aim to make these commissaries pay all expenses in connection with them and make up the deficit, if any, in the boarding houses, thus relieving the job of that burden as much as possible.

General.—In general, this organization worked out well, and there seemed to be very little friction; at the height of the work all departments seemed to be imbued with a spirit of co-operation and a desire to make high records on the concrete work.

Labor.—The labor problem proved very troublesome. Negro labor was used almost entirely, although other classes were tried. The negro, with all his unreliable features, proved the most satisfactory, but had to be treated properly to get the best results. A regular labor agent was employed to handle exclusively the Power Company's work. His headquarters were at the dam, and he reported to the Contractor's General Superintendent. He was a man who knew the Southern labor market well and was thoroughly familiar with all conditions. The greater part of the negro labor came from Florida, and it was necessary to keep a constant stream of men coming all the time. Some negro labor was brought from Mississippi, Memphis, Tenn., and Birmingham, Ala. All transportation was paid by the Company, and deducted later from the laborer's wages. If he remained and worked 60 days, he made his transportation, and it was returned to him.

Quite a few foreign laborers were brought, *via* Savannah, Ga., by sea, from New York City, being secured by New York agencies, but they were not satisfactory at the dam. They were then tried at the quarry and did so much better that for a long period the quarry labor camp was almost exclusively of foreigners. However, the loss was so great on the transportation of foreigners from New York that

labor of this class was finally abandoned, and negroes were put in at the quarries on both day and night shifts. The skilled mechanics, hoist runners, machinists, etc., all came from similar jobs scattered all over the South, and were of the type who follow big jobs of this kind around the country. Quite a few carpenters were brought from New York City, principally Swedes, and proved far more satisfactory than the local talent. The local white labor, skilled and unskilled, was absolutely unreliable, and of an exceedingly poor class. This source did not afford any help whatever in solving the labor question, and was effective in only one section of the work, namely, in floating and placing the coffer-dam cribs. This class was entirely ignored, there being only a very few regular employees drawn from it.

General Organization of the Power Company.—Supervising the Contractor's organization on the power-house foundations and dam, the Power Company's organization was in general as follows: Representing the Chief Engineer directly was the Resident Engineer. His responsibilities under the type of contract used were really those of a General Superintendent of Construction rather than those of an Engineer alone, as not only was he held responsible for the proper execution of the plans, proper inspection of all materials in the field, and the mixing and placing, but also for the working of the job by the Contractor in a proper manner, the disposition of his forces, his expenditures, the proper running and sanitation of his camp, the handling of commissaries, the placing and lay-out of the plant, etc. Furthermore, the Resident Engineer was clothed with full powers to back up and force attention to all orders and instructions given by him in the conduct of the work.

Under the Resident Engineer were two Assistant Engineers, a Chief Inspector at the quarry, and an Inspector of the railroad; one Assistant Engineer looking after the details of the office and all the night work in conjunction with the Resident Engineer, and all such general details of the job as the Resident Engineer turned over to him for attention. The other Assistant Engineer, with his assistants, not only looked after the proper handling and placing of the concrete, but at all times kept close track of the Contractor's working organization on the dam to see that it was handling the work efficiently. He conferred constantly and worked almost entirely with the Contractor's Superintendent on the dam, his main duties being to look after the construction

on the dam and power-house only, to see that all features were according to the specifications, and that the forces were properly organized and handled.

A day and night inspection was maintained at the quarry, the Chief Inspector handling the day shift and the inspection work on that shift. He was assisted by a timechecker, clerk, and night inspector. He was held responsible for the proper inspection of all crushed and cyclopean stone, and saw that only clean materials were loaded. He watched the loading at the quarry, saw that all stone was washed before going to the crushers or to the dam as cyclopean, and measured all cars of crushed stone before they were pulled out. He also was held responsible for the proper checking of all materials and labor used by the contractor there, and reported any inefficient handling of men or equipment. The Night Inspector performed the same duties at night, reporting to the Chief Inspector. The Power Company's forces were provided with a comfortable house and office entirely separate from the Contractor's forces. They, however, took their meals at the Contractor's mess hall.

The Inspector of the Railroad performed the same general duties on the railroad as other inspectors, and his assistant also checked time and materials, as already outlined. His duties, however, were not as important as those of the other inspectors.

A force of time and material checkers, entirely independent of that of the Contractor, was maintained in each camp, and vouchers and pay-rolls were approved only on the Power Company's check. All expenditures for material and labor had to receive the approval, in writing, of the Resident Engineer, and this approval was essential to the passage of any voucher for payment.

General day and night forces of inspectors, checkers, timekeepers, etc., were maintained by the Power Company at the dam. All the day inspection work was performed under a Chief Inspector, and he, with his inspectors at the dam and at the mixer, checked all forms, looked after the proper mixing, placing, and working of concrete, the proper preparation of the surface of old concrete, saw that all bolts and reinforcing were properly set, etc. The inspectors on the dam and power-house gave orders to the concrete foremen and walking bosses where they concerned the proper working of the concrete and the cleaning of the surface of old concrete before placing new concrete

on it. They had the authority to prevent any of the Contractor's men from commencing to place concrete if the forms were improperly cleaned out. Great stress was laid at all times on the proper cleaning of the concrete. The mixer inspectors watched the proper proportioning of the mix and the quantity of water necessary, receiving their instructions from the inspectors on the dam. At the mixing plant, there was also a cement checker on each shift.

The night shift organization was handled in the same manner, with an Assistant Engineer in general charge for the Power Company, and a Chief Night Inspector under him. All the survey and lay-out work was handled by two small parties of two or three men each. One of these parties was assigned to the dam and another to the power-house. All levels, lines, grades, etc., were given by these parties. Before and after forms were set up by the carpenters, the lines and levels were given by one of these field parties, and the inspectors would allow no concreting in a form until it was "O. K'd." as to grade and line by the Chief of the Party.

This system of handling the inspection and survey work proved most satisfactory, and is recommended for similar work.

Power-House Superstructure and Equipment.

As previously mentioned, the construction of the power-house superstructure and the installation of machinery was done by the Company's own forces. In October, 1913, the organization of the Company's construction forces was begun, most of the men being taken from Gadsden, where, as noted early in the paper, a 10 000-kw., steam turbo-generating station had been constructed.

A camp was constructed near the main camp of the Contractor's forces, and independent mess halls were provided. Storage yards for structural steel, brick, machinery, etc., were constructed part way down the Low-Level Line, and a traveling gantry crane, later erected in permanent position on the up-stream side of the power-house, was placed in these yards for the purpose of unloading and storing heavy machinery parts.

Steelwork.—On November 4th, the foundations were in such shape in the power-house that steel erection was started. A spur track was built from the Low-Level Line in the bottom, entering the west bay of the power-house on a trestle at Elevation 370, approximately, it being

planned to bring all steelwork, machinery, etc., in on flat cars here and remove it with first a locomotive crane and later with the permanent 100-ton crane. A 30-ton locomotive crane was set up inside the power-house for the purpose of erecting all columns and cross-girders. This crane had a 50-ft. boom with a 30-ft. extension. On Figs. 45 and 46 this crane is shown at work erecting columns, and also the track on which all steel and other materials were brought in. As soon as the steel was in condition to receive the 100-ton crane, it was erected, and the locomotive crane was withdrawn for the purpose of loading cars in the yards. A traveling stiff-leg derrick was then erected on the heavy cross-girders above the 100-ton crane, and it handled the remainder of the steel erection, the 100-ton crane taking the steel from the cars below and delivering it to points under the traveling derrick.

Brickwork.—After riveting was sufficiently advanced, the brickwork was started on a previously moulded concrete water-table. All the window sills, cornices, and ornamental work were of concrete blocks, moulded and cast on the work. All concrete block forms were of cast iron, and the results obtained were very good. Gravel and coarse sand were used for making the concrete. They were mixed fairly dry, being well packed and rammed into the moulds.

Erection of Units.—The foundation rings and shell of the units came in sections. These sections were loaded by the 30-ton locomotive crane on flats and delivered to the 100-ton crane in the power-house. These sections were then assembled on the floor adjoining the foundation on which they were to be set, and put in place, properly lined and leveled up, grouted, and then concreted. The type of speed ring and casing used permitted it to be set directly on the foundation previously prepared. It was blocked into permanent position by jacks and holding-down, foundation blocks, and then solidly concreted. The first speed ring was set for No. 4 Unit on January 8th, 1914, and this unit went into commercial service on April 12th, 1914, the others following rapidly thereafter. The first unit was operated through a temporary switch-board and transformer-house set up outside the main building, in order to start selling this power at the earliest possible moment. This allowed the setting up complete of the main busses, switch-boards, and wiring, without current being on them. Later, when these three units were all installed and everything in readiness, the unit that

had been operating temporarily was pulled out of service, the transformers put in their permanent locations, and the few connections between the unit and switch-board easily and quickly made.

Gates.—Penstock gates were set at the time the units were being prepared for service. Some trouble was experienced on all gate frames on account of the castings being too light and warping or springing. This was true of the spillway gates and the guides for the screens in front of the penstock gates. It is almost impossible to block these to exact line, and, in the cases in question, the clearances were entirely too small. Also, the method of setting gate frames in the original pour of concrete around them should be avoided. In view of the experience gained on this work the writers recommend strongly that in similar designs the following points be observed in the design and erection of cast or structural gate frames:

1. See that ample clearances are allowed, giving wide bearing faces to make up the excess width allowed.
2. All faces should be machined true and square, and should be carefully inspected with that end in view at the mill or factory.
3. Grooves and pockets with proper anchorages should be left in the concrete, the frames should be set up after the mass of concrete is poured, and then carefully lined, anchored, and grouted in place.

A gantry crane which traveled on the front side of the power-house, at Elevation 425, handled and set all the large penstock gates. These gates weighed about 12 tons each, and were operated by large cylinders, using oil under pressure. The spillway gates were all assembled and riveted in the assembly yard, some distance from the dam. They were then loaded on cars and delivered to a scow, built for the purpose, just above the dam. They were towed to the front of the openings, and put in place. These gates were operated by a traveling car which, through a flexible coupling, engaged a gear operating the hoisting mechanism of each gate. This traveling car was in duplicate, one electrically operated, the other steam-driven.

Reservoir Clearing.

Of the 4 700 acres of reservoir land, 1 700 acres were more or less heavily wooded. A thorough investigation showed that there would

be no actual ill effects to the health of the community at large, by leaving this timber standing, but the temperament of the hill people residing in the neighboring territory, and their respect for the law, in so far as it may assist them in obtaining compensation for imagined damages, made it appear prudent to clear the reservoir. This decision was strengthened by the fact that a power company in Georgia, which had not cleared its reservoir, was having endless legal trouble from damage suits filed against it. After consultation with the health officials of the State, it was decided that no timber should be left standing, or in a fallen position, between the 420-ft. contour, the normal lake level, and the 410-ft. contour, the limit of the draw-down. Wherever merchantable timber was within a reasonable distance of the railroads, it was sold, and a quantity of pine timber was converted into charcoal; whatever could not be disposed of in either way was burned on the ground, if possible.

Transmission Lines.

The standard line construction adopted by the Company involved the transmission of current at 110 000 volts and distribution at 22 000 volts. For the former, the double-circuit, galvanized-steel towers were adopted. These towers carry two ground wires of $\frac{3}{8}$-in. Siemens Martin, and six No. 00 copper, 7-strand conductors, drawn to 55 000 lb. per sq. in. in tensile strength. The towers were of three types: Type A used in rough country, weighs 4 600 lb., is 68 ft. high, and has a 10-ft. spacing of conductors vertically and 15-ft. horizontally on the top and lowest arms, and 16 ft. 6-in. on the intermediate arm. The base of this tower is 15 ft. square. Type B towers were used as strain towers; they weigh 4 700 lb. and are of the same general design as the other types except that they have bases 17 ft. square; these towers are used at angles between 3° and 35° and at every $2\frac{1}{2}$ miles on the lines. The Type C tower is used in flat country in connection with Type B towers, the latter being placed at each mile point. These towers weigh 3 600 lb., are 65 ft. high, and have bases 16 ft. square. The ground clearance allowed was 25 ft., and the towers were located so that with a $\frac{1}{4}$-in. coating of ice and 8 lb. wind pressure, this clearance would be maintained. The normal spacing for Type A towers is 700 ft., and for Type C, 600 ft.; the maximum allowed spacing for Type A is 1 200 ft., and for Type C, 900 ft.

The insulators used on the 110 000-volt lines were of three makes: the Ohio Brass, Thomas, and Locke. The strings were made up of 6 units on suspension and 7 on strain. The disks are 10 and 12 in. in diameter. The 6-disk strings were tested to 350 000 volts dry and to 255 000 volts wet, before flash over. The test for mechanical strength consisted of a pull of 5 000 lb. applied in the direction of the axis.

The right of way secured for transmission lines was 100 ft. wide. The towers were set on the middle of this strip, and, 10 ft. from the edge, there was carried an all-copper telephone circuit of No. 8 hard-drawn wire, on 22-ft. creosoted poles, 175 ft. apart. On the telephone lines, porcelain insulators, designed for 6 600 volts, were used on a single cross-arm. The line is transposed every fifth pole.

Wagons were especially designed for the transmission line surveys, and their equipment was very compact. Each wagon contained sleeping accommodations, and cooking and table utensils for a party of thirteen men. A negro cook and a negro who acted as driver and general utility man were carried with the party.

The sequence of construction was as follows: When the right of way was secured, a clearing gang, normally consisting of sixty men and two foremen, cleared the line. Immediately following the clearing gang was the telephone gang, consisting of a foreman and twelve men. This gang erected poles and strung the line complete, tying into the public long distance telephone lines at convenient points, so that at all times there was easy communication with the main office. Following a short distance behind the telephone gang was the foundation gang of seventy-five men and three foremen. This gang dug holes, set the stubs of the towers, and back-filled, or set the rock-anchors, as the case might be. The assembling and erecting gangs of forty men total and two foremen for the two gangs, followed as closely as convenient. The stringing gang, of sixty men and three foremen, hung insulators, and strung the ground wires and conductors, finishing the line ready for service.

Fig. 50 shows part of the stringing gang at work on the line from Lock 12 to Anniston, the right of way being occupied by the tower line and a pole line carrying 22 000 volts. Fig. 51 shows the Sylacauga-Alexander City, 22 000-volt line. In this line, the conductors were carried on wishbone-type cross-arms, and the ground wire on

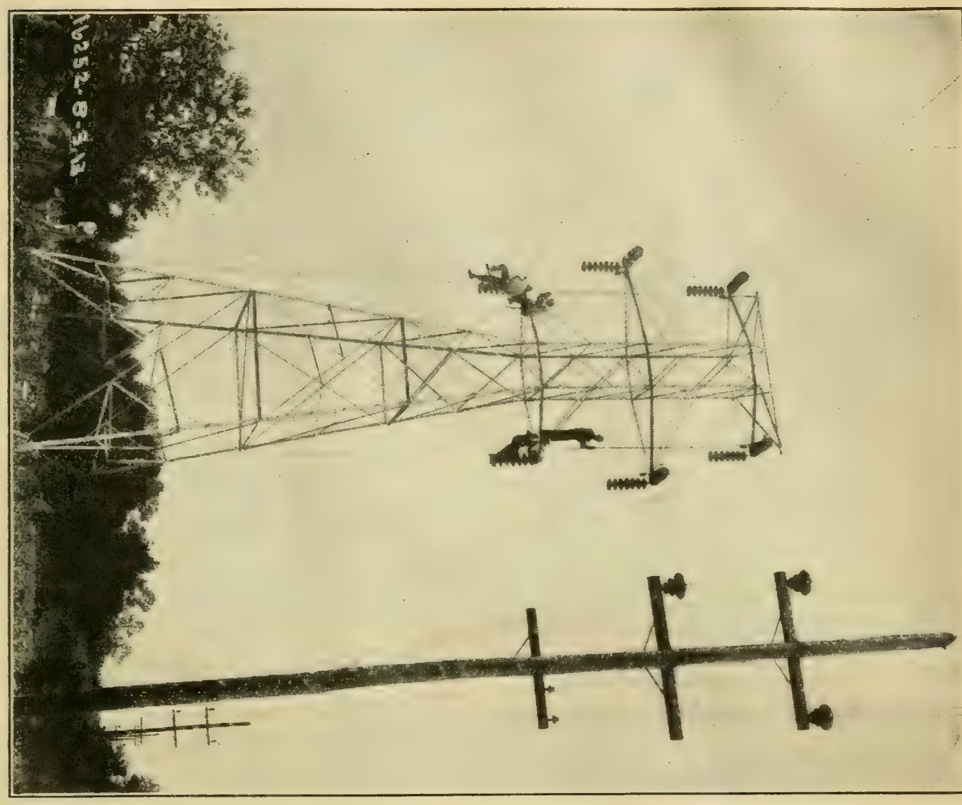


FIG. 50.—STRINGING COPPER CONDUCTORS ON TRANSMISSION LINE OF 110 000 VOLTS.

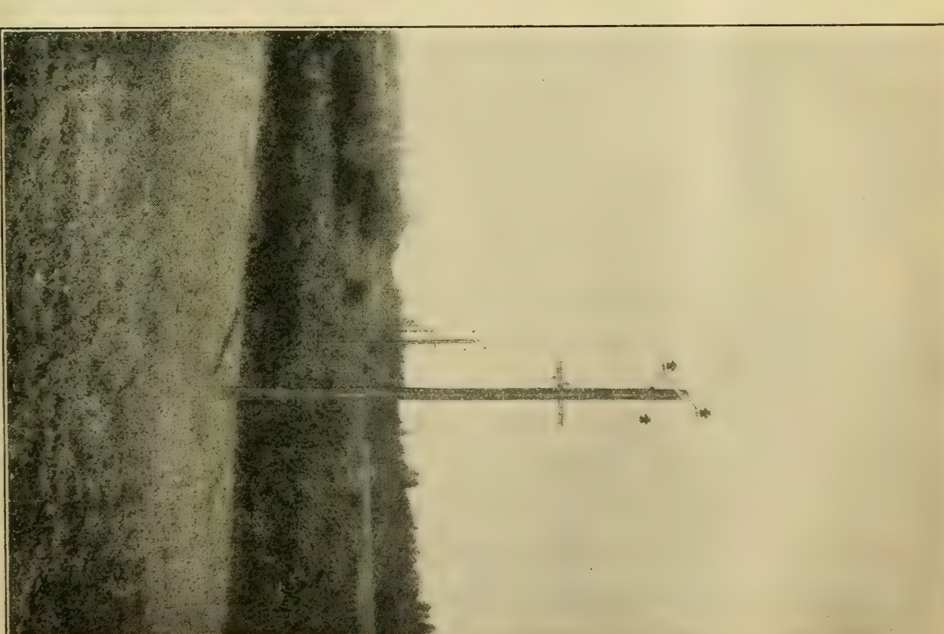


FIG. 51.—DISTRIBUTION LINE FOR 2 000 VOLTS, USING WISHBONE TYPE OF CROSS-ARM.

a bayonet. On all 22 000-volt lines, the telephone line was carried on the same pole with the conductors.

Electrolytic lightning arresters were installed on all lines at the generating stations and at sub-stations.

Sub-Stations.

Sub-stations have been built at Anniston, Jackson Shoals, and Gadsden, and one is under construction at Magella, on the outskirts of Birmingham. The capacities of these sub-stations are as follows:

Anniston	6 000	kv-a.
Jackson Shoals	6 000	"
Gadsden	12 500	"
Magella	40 500	"

The outdoor-type of sub-station was selected because the housing of 110 000-volt apparatus would have been very expensive, due to the large clearances which are necessary with this voltage.

The Anniston Sub-station is a switching station for the two 110 000-volt lines which pass through it. The equipment includes three 2 000-kv-a., single-phase, oil-insulated, self-cooled tubular-type transformers, stepping the voltage down from 110 000 to 22 000 for distribution in the Anniston district. The efficiencies of these transformers at full, three-quarters, and one-half load, are 98.4%, 98.2%, and 97.7%, respectively. A small brick building, Fig. 53, houses the switchboard, three 667-kv-a., 22 000 to 2 300 transformers for local duty distribution, the storage batteries for operating the oil-switches, and a motor generator set for supplying direct current to the Anniston Street Railway.

The Jackson Shoals Sub-station is a switching station for two 110-kv. from the north and south and one 110-kv. from the west. At this point, the voltage is stepped down to 22 000 for distributing lines to the north, south, and west. The equipment includes three 2 000-kv-a., oil-insulated, water-cooled, single-phase transformers, with efficiencies of 98.05%, 97.9%, and 97.5%, at full, three-quarters, and one-half load, respectively. Near the sub-station is the small hydro-electric plant generating at 2 300 volts. A bank of transformers steps this voltage up to 22 000 volts, and the plant is tied into the 22 000-volt bus of the sub-station.

The Gadsden Sub-station is a distributing as well as a step-up station. The generating voltage of the steam plant is stepped up from 2 300 volts to 110 000 or 22 000 volts, or the 110 000 voltage of the transmission lines is stepped down to 22 000, as the case may be. The equipment of the station includes two banks of three transformers each, and a spare-all single-phase. oil-insulated, and water-cooled—each of 2 100-kv-a. capacity. These transformers have three windings, for 2 300, 22 000, and 110 000 volts. The guaranteed efficiencies at full, three-quarters, and one-half load are 98.3%, 98.1%, and 97.6%, respectively.

The Magella Sub-station is a distributing station for three 110 000-volt lines, two from the south and one from the east. The station will step down from the 110 000 volts of the transmission lines to 22 000 volts and to 13 200 volts for distribution, the latter being for distribution into Birmingham. Provision has been made for increasing the capacity of this station from 40 000 to 67 000 kv-a. The equipment includes three banks of three transformers each, and a spare-all single-phase, 4 500-kv-a. capacity, oil-insulated, and water-cooled.

Fig. 52 shows the bus structure of the sub-station at Anniston, which is typical of the others. It is built of towers and girders, this construction having been found to be more economical than an arrangement of towers alone. All the material in the structure is galvanized steel. The weights of the structures are: Anniston, 22 tons; Jackson Shoals, 35 tons; Gadsden, 30 tons; and Magella, 50 tons.

The 110 000-volt busses are No. 00, B. & S. gauge, copper wires, supported horizontally by seven, and vertically by six, 10-in. disk insulators. The busses are 8 ft. apart and are 5 ft. from the nearest steel.

The oil-switches are of the out-door, solenoid-operated, remote-control type, operated from storage batteries in the switch-houses.

Provisions are made at each sub-station for the repair of transformers. There are electrolytic lightning arresters on all 110 000- and 22 000-volt lines, as a protection against the severe electric storms of the region.

It is proposed to standardize future installations of sub-stations into capacities of 3 000, 6 000, and 10 000 kv-a. The bus structure, being made up in unit panels, will be adapted readily to this standardization.

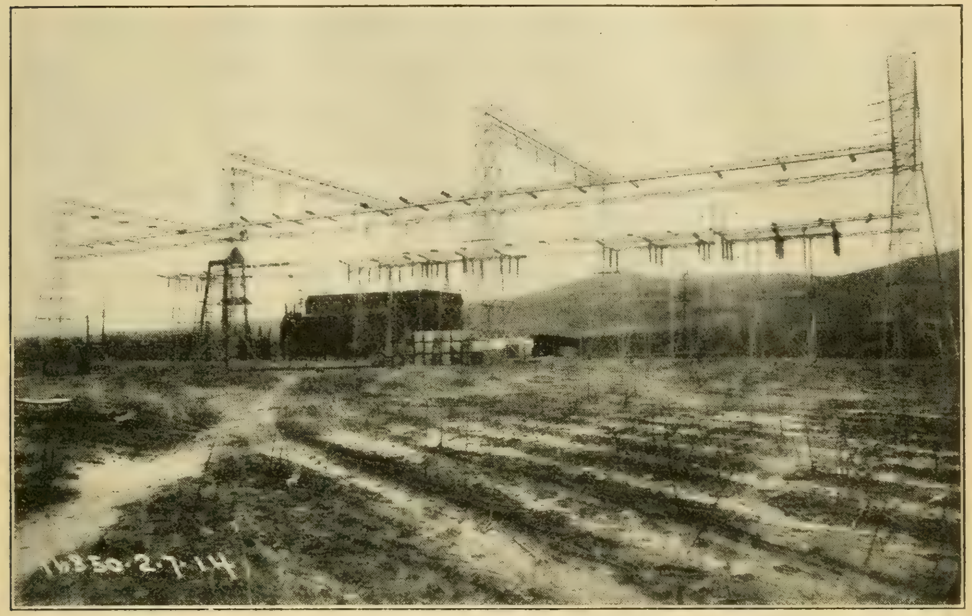


Fig. 52.—Outdoor Sub-Station at Anniston, Ala.; Construction is Typical of all Sub-Stations.

Fig. 53.—Interior of Switch-House at Anniston Sub-Station.

Organization.

All this work was done under the supervision of E. A. Yates, Assoc. M. Am. Soc. C. E., Chief Engineer; E. L. Sayers, M. Am. Soc. C. E., was Assistant Chief Engineer, O. G. Thurlow, Designing Engineer, W. E. Mitchell, Electrical Engineer, A. C. Polk, M. Am. Soc. C. E., Resident Engineer at Lock 12, W. C. Cram, Jr., Superintendent of Transmission Line and Sub-station construction, and S. B. Jones, Superintendent of Construction at Lock 12 Power-House and Gadsden Steam Plant. MacArthur Brothers Company were the Contractors for the power-house foundations and dam.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE GAUGE OF RAILWAYS, WITH PARTICULAR REFERENCE TO THOSE OF SOUTHERN SOUTH AMERICA.

Discussion.*

By Messrs. T. A. Corry, Frank Foster, H. Deane, and G. F. F. Osborne.

T. A. Corry, M. Am. Soc. C. E. (by letter).—Mr. Lavis' figures show that more than 60% of the Argentine Railways are broad-gauge, and only about 8% are standard. This fact, taken in connection with the lack of ballast and consequent difficulty in keeping the track in good surface, proves unquestionably that if only one gauge is to be adopted, the broad-gauge is the proper one for the Argentine, for Mr. Lavis is quite right in his remarks on the relative stability of broad- and narrow-gauges. Mr. Corry.

For such mountainous countries as Bolivia, Peru, Ecuador, Colombia, and Venezuela, the advantages of the standard gauge over the meter-gauge may be questioned for three reasons:

First. With the meter-gauge the capacity of 95% of the railways required, for the last 30 years and the next 30 years, to handle the available traffic would be quite ample.

Second. Sharper curves can be used on narrow-gauge than on wide-gauge for the simple reason that the rigid wheel base is shorter on both cars and locomotives for the narrower gauge, and the possibility of using sharper curves very greatly reduces the graduation item—a heavy one in traversing the slopes of the Andes, on either the west or east side, in the countries mentioned.

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* Continued from April, 1914, *Proceedings*.

Mr. Corry.

Third. It is good business to use a small gun and small shot for small game, especially when expenses are reduced thereby.

It is axiomatic, of course, that a considerable amount of traffic, even in a mountainous country, can be handled at less cost per ton per mile on a standard gauge than on either of the narrower gauges, and the standard is the more economical even if it does cost more to construct.

As to speed, on curves of 100-m. radius (328 ft.), the safe limit is about 28 miles per hour, with the practical elevation slope of 6°, and this speed can be made without difficulty on either 3-ft. or meter-gauge. To refer to a concrete example, take the section of the Peruvian Southern Railway from Mollendo, on the Pacific, to Puno, on Lake Titicaca (Plate XXIX). This road is of standard-gauge, 326 miles in length, and was built in 1868-73, at a cost of 42 000 000 soles (at that time the sol was worth 48 pence, or, say, 97 cents). The maximum gradient was 4%, and the minimum radius of curve 100 m. (328 ft.), and the minimum was used very frequently.

The Mollendo Division is 172 km., or 107 miles, in length, and cost 12 000 000 soles. Besides numerous short tangents, it has six of from 2 to 7 miles in length, aggregating 25 miles. On the remaining 82 miles there are 421 curves, an average of 5 curves per mile. More than half these curves are of only 100-m. radius with an angle per mile of more than 250 degrees.

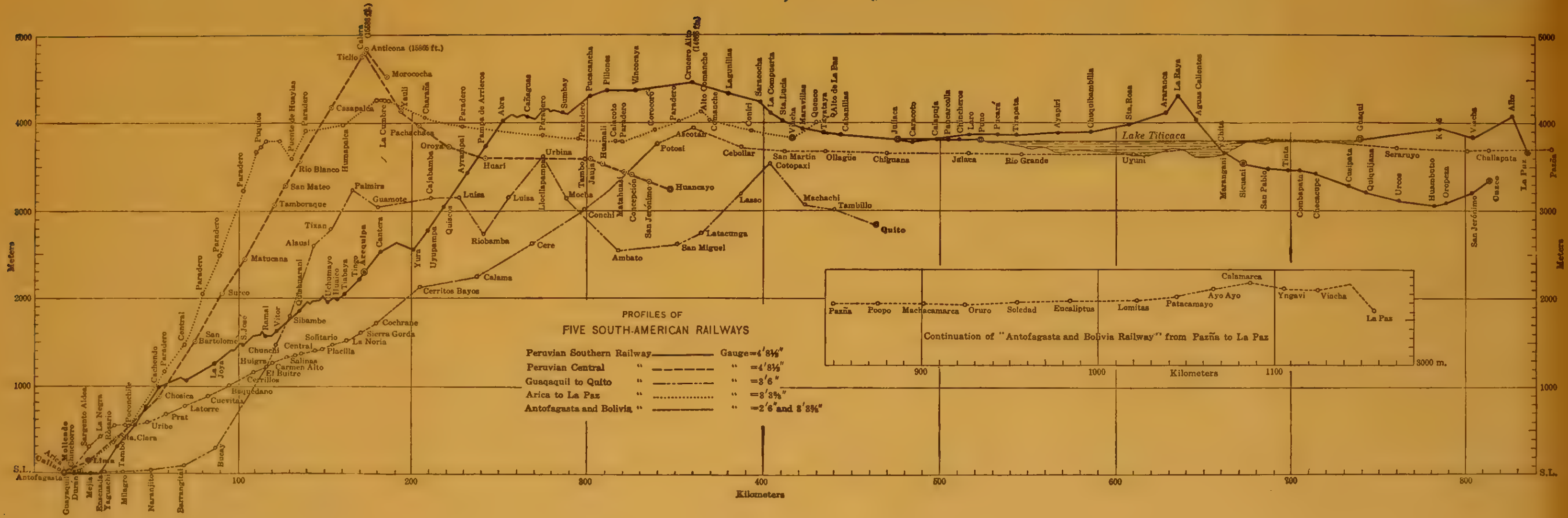
The Puno Division is 352 km. (219 miles) in length, and cost 30 000 000 soles. It has twelve tangents varying in length from 2 to 12 miles, comprising 46 miles, exclusive of those of less than 2 miles in length. The curve section is 173 miles in length and has 783 curves, an average of 4½ curves per mile, the larger number of which are on rough ground, and principally solid rock.

The total daily average tonnage, both ways, has never exceeded 600 in any of the 41 years of the road's existence. The average haul is large, being some 200 miles.

Whether it might have been advisable to have built this road with a 3-ft. gauge, using curves of only 80 m., is debatable on the ground of smaller first cost.

To give another illustration: A few years ago the writer was instructed to make a survey from a point on this road to a river of the Amazon Valley, less than 300 miles in length. The bases given were: maximum gradient, 3%; minimum radius of curve, 120 m.; and gauge, 3 ft. On these bases, the estimated cost reached the sum of $12 500 000, which was a prohibitive one for the traffic and advantages expected.

A resurvey of the roughest part (60 miles), using a gauge of 30 in. instead of 3 ft., with a minimum radius of curves of 50 m.


PROFILES OF
FIVE SOUTH-AMERICAN RAILWAYS
Peruvian Southern Railway — Gauge=4′8½″
Peruvian Central " =4′8½″
Guaqaquil to Quito " =3′6″
Arica to La Paz " =3′3⅜″
Antofagasta and Bolivia " =2′6″ and 3′3⅜″
Continuation of "Antofagasta and Bolivia Railway" from Pazña to La Paz
Meters
Kilometers
Lake Titicaca
Anticona (15865 ft.)
Crucero Alto (14666 ft.)
Quito
La Paz
Arequipa
Huancayo
Lima
Callao
Antofagasta
Arica
Guayaquil
Cuzco
Puno
Juliaca
Oruro
Uyuni
5000
4000
3000
2000
1000
S.L.
100
200
300
400
500
600
700
800
900
1000
1100
3000 m.

Mr. Corry

and a gradient of 5% (for use with Shay locomotives), resulted in a reduction of $3 500 000, or nearly 30 per cent. The principal reduction was due to the sharper curves, by which much heavy work and all the tunnels were avoided. The expected traffic would hardly reach 300 tons per day for the next 20 years.

The Antofagasta and Bolivia Railway Company is changing its 30-in. gauge to 1 m., so that now there are in Bolivia somewhat more than 800 miles of railways—all of 1-m. gauge.

In 1912, the total tonnage, including company material, handled was 1 600 710, or nearly 4 400 per day, the average distance hauled being about 130 miles. In view of the great expense being incurred by this road in changing its gauge to 1-m., it is to be presumed that the relative merits of 1-m. and standard-gauge have been carefully considered.

While the writer is not an advocate of narrow-gauge railways, it must be admitted that there are still some mountainous districts, where their use, especially as feeders, is advisable, even if it should prove necessary after a number of years to widen the gauge, and possibly at the same time make other improvements in the matter of alignment, gradients, etc. To put it another way, it may often be inadvisable to build now, for traffic to be handled, say, 40 years hence.

Mr. Foster.

Frank Foster, M. Am. Soc. C. E. (by letter).—In his able paper on railway gauges in South America, Mr. Lavis puts too lightly to one side what appears to the writer to be the principal advantage of the narrower gauges.

The Argentine broad-gauge (5 ft. 6-in.) will not allow of shunting with main-line engines on curves of 200 ft. radius, and requires turn-outs with curves of 600 ft. radius for goods trains; and curves well maintained of less than 1 600 ft. radius are rough for main-line running.

In comparison, the meter-gauge stock can circulate on curves of 120 ft. radius, can use turn-outs with curves of 250 ft. radius for goods service, and curves well maintained of 1 000 ft. radius can be taken comfortably at full speed on the main line.

The writer does not wish to imply that broad-gauge stock could not be constructed to circulate under these conditions, for they certainly could, but always at the sacrifice of velocity.

Mr. Lavis quite correctly points out that the broad and standard gauges have enormous advantages in speed over the narrow gauges, but, in fairness, this principal advantage of the narrow-gauge, namely, normal velocity on curves of smaller radius, should be equally extolled. As this is, in the writer's opinion, the unique advantage of the narrow gauges, their use in the Argentine is only justified in mountainous districts, and is to be deplored in the plains, for, without any financial or physical advantages, they have bred costly transhipment

Mr. Foster. which can only be avoided by a duplication of expenditure, frequently unjustifiable.

Unfortunately, the original railways in the Argentine were constructed of the 5 ft. 6-in. gauge in preference to the standard 4 ft. 8½-in.; and although any idea of correcting this error now or in the future would be financially unsound, as the advantages could in no way compensate the outlay, yet the writer is of the opinion that Mr. Lavis in no way exaggerates the advantages of the standard-gauge as compared with those of the broad-gauge.

An examination of the minimum construction and maximum rolling-stock gauges of the broad-gauge of the Argentine (Fig. 3) shows them to be very similar in dimensions to those of the standard-gauge in the United States, which indicates that in the Argentine no use is made of the wider gauge.

On several occasions the advisability of enlarging the rolling-stock gauge has been under discussion between the officials of the railways and the Argentine Government Railway Board, but it has been resolved, for the reasons given by Mr. Lavis, that there was not sufficient advantage, either in locomotives, coach, or wagon stock, even though the cost of increasing the construction gauge was not prohibitive. For

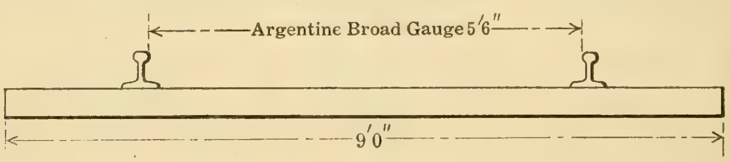


FIG. 13.

instance, the broad-gauge lines of the Argentine at that time had no tunnels, and there are very few over-bridges.

The carrying capacities of the Argentine broad-gauge and the United States standard-gauge are in reality the same, the only appreciable difference between them being the position of the rails, that is, the gauge.

The rail position of the Argentine 5 ft. 6-in. gauge, on 9-ft. sleepers, Fig. 13, represents a very distinct disadvantage to maintenance, as the length of the sleepers outside the rails is not sufficient for adequate support; theoretically, the maximum pressure transmitted by these sleepers is at their extremities instead of under the rails, as is the case with standard-gauge and 9-ft. sleepers, and is at that point 35% in excess of the average pressure instead of 10 per cent. These disadvantages exist, to the writer's practical knowledge, and, in earth ballast, cause excessive pumping at the extremities after rains, and frequent breakage of the sleepers at the center.

In this respect the Argentine meter-gauge tracks, with 6-ft., and in some cases 6 ft. 8-in., sleepers, have better proportions, but the overhang of the rolling stock, shown by Fig. 3, is out of proportion—for

good maintenance on earth ballast—both to the gauge and to the sleeper length. This practically results in greater expenditure in maintenance, rougher riding, and reduced velocity, as compared with the broad gauge. Owing to the general absence of stone in the plains, the majority of the railways in the Argentine are earth-ballasted, except where the traffic is dense or in the few districts where stone exists. Mr. Foster.

The ratios between the rolling-stock gauge, length of sleepers, and gauge of track are given in Table 30.

The sleepers used in the Argentine are of a specially hard native timber (*Quebracho colorado*), of a density of 80 lb. per cu. ft., which permits the use of a dog-spike connection. These sleepers have a life equal to that of the rails they carry, in many cases exceeding 35 years.

Dense forests of these trees grow in the Northern Provinces of the country, and the soundest timber for sleepers is obtained from the branches sawn in two. The sawn side is laid downward for the bearing on the ballast and the upper side is adzed, as they are rough in section and all the sap-wood rots away entirely within a couple of years. A typical section of one of these sleepers, after the loss of the sapwood, is shown in Fig. 14.

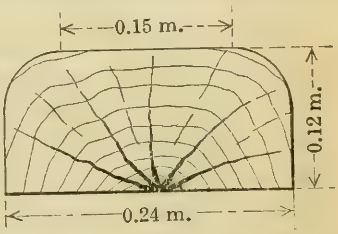


FIG. 14.

Sleepers can be obtained square sawn from the main tree trunk, but are not to be preferred as they frequently split on exposure or spiking. Consequently, up to certain lengths, the cost per cubic foot is more or less constant, but for a length of more than 9 ft. the cost increases considerably.

As there are no native timbers other than the quebracho and similar hardwoods available in the Argentine, apart from the fact that legislation actually calls for the use of the timber of the country, everything is in favor of the use of quebracho in preference to any other timber on these railways.

TABLE 30.—Ratios Between the Rolling Stock, Gauge, Length of Sleepers, and Gauge of Track.

	Track gauge.	Sleeper length.	Rolling stock width.	Ratio of sleeper length to track gauge.	Ratio of rolling stock width to track gauge.	Ratio of rolling stock width to sleeper length.
Argentine: Broad-gauge.........	5 ft. 6 in.	9 ft. 0 in.	11 ft. 2 in.	1.63	2.03	1.24
Argentine: Narrow-gauge........	3 ft. 3⅜ in.	6 ft. 0 in.	10 ft. 6 in.	1.83	3.20	1.75
United States: Standard-gauge..	4 ft. 8½ in.	8 ft. 6 in.	10 ft. 3 in.	1.80	2.18	1.21
United Kingdom: Standard-gauge	4 ft. 8½ in.	9 ft. 0 in.	9 ft. 0 in.	1.91	1.91	1.00

Mr. Foster. Therefore, the Argentine broad-gauge lines will probably continue to use sleepers of this length despite the serious disadvantage to maintenance indicated by the ratios given in Table 30.

On the level plains of the Argentine, curvature has not caused serious inconvenience thus far, but the waste of space in valuable terminals by the length of one-in-eight turn-outs (85 ft.) is an additional serious defect in the broader gauge, whereas, in the writer's opinion, there are no advantages of any kind in the use of the 5 ft. 6-in. gauge in the Argentine, as compared with those of the 4 ft. 8½-in. gauge.

The writer has not entered into the question of methods of transportation, but would point out that on the Buenos Aires Western Railway it is usual to run trains of more than 2 000 tons gross weight with locomotives weighing more than 100 tons. These trains at times attain a length of 2 500 ft. On that railway, wagons with carrying capacities of from 40 to 45 tons represent more than 80% of the total carrying capacity of those in service.

Mr. Deane. H. Deane, Esq.* (by letter).—The writer has been much interested in following the arguments set forth by the author, and believes that he has arrived at the right conclusion when recommending for adoption what has practically become the standard and world gauge. The conditions pertaining to the problem in Australia differ somewhat from those in South America, in that the interests of the Australian States are really the same, and there is a tendency toward one or at least a similar kind of control in all matters. In the case of the railways there have taken place, for many years past, annual meetings of the officers and Commissioners who have charge in each State, and the desire always has been to work for one common object, the good of all. In South America there are many independent States, the Governments of which are not actuated by any desire for the common good; at the same time, it must be felt that individual interests would best be served if there were one gauge common to the whole country, for when the railways meet on the borders, interchange would thereby be facilitated.

In spite of what has been said in a few quarters as to the inadequacy of the 4 ft. 8½-in. gauge to meet all conditions of crowded traffic, it must be recognized that a medium gauge suitable for the vastly preponderating ordinary conditions should be adopted—one that is not so wide as to be prohibitive in cost when used in a country where the population is sparse and traffic is light. The 4 ft. 8½-in. gauge has been proved to meet all requirements fairly, and it is doubtful whether much more could be done with any much wider gauge than is achieved in the United States on the 4 ft. 8½-in.,

* Recently Engineer-in-Chief for Commonwealth Railways. Melbourne, Australia.

seeing that to get the full theoretical advantage of very much wider gauges, rolling stock would have to be built of very unwieldy dimensions, and this very unwieldiness would tend to reduce its economic value. Mr. Deane.

The case of India seems to show that a gauge much wider than 4 ft. 8½-in. would sooner or later prove a mistake. Railway construction was commenced in that country on a 5 ft. 6-in. gauge, but after a time that gauge seems to have been found to be inconveniently wide, and recourse was had to the meter-gauge for large extensions. The mileage of the railways on this gauge seems to be about 40% of the whole system. Had the 4 ft. 8½-in. gauge been adopted at the outset, there would probably have been no meter lines. Mr. Lavis has shown that, where the meter-gauge was thought necessary, the 4 ft. 8½-in. gauge meets the conditions quite as well. The writer proved that himself when constructing the Wolgan Valley Railway in New South Wales, and this has been mentioned by the author.

In selecting a gauge as a standard in any country, it should of course be borne in mind, that sooner or later connections will be made with outside countries, when all the evils of breaks of gauge will be experienced if the matter has not been previously adjusted.

There is one argument in favor of the 4 ft. 8½-in. gauge which should not be lost sight of. The present tendency is to adopt the same standards for mechanical details throughout the world, so that when an article is ordered from a distance there may be a certainty that it will fit into its place. The chief manufacturing countries—United States, Canada, Great Britain, and those of the continent of Europe—are engaged in building locomotives and other rolling stock to the 4 ft. 8½-in. gauge, and it is clear that if, in a certain country, there is a sudden demand for rolling stock which cannot be met out of its own resources, orders placed in any of the countries mentioned could be much more promptly met if the country ordering uses the same gauge; whereas, with a different gauge, there would necessarily be special designs, special patterns would have to be made, and so on.

G. F. F. Osborne, Assoc. M. Am. Soc. C. E. (by letter).—The writer has been connected with the administration of a railway comprising a section of 5 ft. 6-in. gauge adjacent to a large port, and a meter-gauge section, farther away, built as an extension of and as a feeder to the 5 ft. 6-in. section. Although the accounts and statistics of the two gauges were kept separately, both were worked as one system by one establishment of administrative and executive officers, and members of the staff in all grades were transferred, as occasion arose, without reference to the section on which they were serving or to serve. The results, therefore, afford a very fair basis for judging the relative operating merits of broad- and narrow-gauge for the conditions there obtaining. Mr. Osborne.

Mr. Osborne.

Both lines are built over flat country intersected with waterways. There was very little cutting (practically none) on either system. The area served by the broad-gauge is densely populated; that served by the narrow-gauge more sparsely so. In both cases the individual inhabitant has a very small traffic value, either by his personal journeys or as a producer or consumer.

Before these lines were built, the country was served by a well-organized system of water carriage, which still exists; and competition is severe. The system has arrived at its present state partly by direct construction, but partly by the absorption, by purchase, of other lines; and their purchase price is now shown in the capital accounts as a lump sum. The broad-gauge section bears the brunt of the expenses due to terminals, has a heavy suburban traffic, and is generally constructed on a higher standard than the meter-gauge. On the other hand, the possible extension of the broad-gauge to replace the meter-gauge (now in progress over some 60 or 70 miles) has led to the construction of bridges having spans of 60 ft. and greater on the broad-gauge standard, and the expense (a considerable item) is charged to the meter-gauge. Another broad-gauge railway reaches the port by running powers over the quadruple portion of the broad-gauge section and two of the four tracks are necessitated by this traffic (which has not been here considered). The meter-gauge stops about 125 miles from the port, and all through traffic is transhipped at the point where the gauge changes.

The system is a network rather than a line, and has many branches. The longest distance run by a through train is 150 miles on the broad-gauge and 335 miles on the meter-gauge. Over the whole system the average haul of a passenger is 23 miles; of freight, 192 miles. Practically the whole staff is housed in buildings belonging to the administration and charged to the capital of the undertaking. Station buildings, houses, platforms, and similar structures are of masonry. The system is soundly built, well equipped, and adequately maintained.

Practically all the passenger stock is fitted with automatic brakes, but this is true of only a small percentage of the freight stock. The busier parts of both sections are worked on the block system, with token instruments; the remainder is worked on the written "line clear, authority to proceed" system. Signaling is normal danger.

The statistics in the tables are for 1912, and are derived from the half-yearly analysis of working the system; the sum or mean of the figures for the two half-years being taken. The error (if any) thus caused is not sufficient to affect the comparative value of the figures.

Before considering the operation statistics, it may be stated that the meter-gauge is being extended through country where the line is not expected to pay in itself for some time after opening, and profits have to be looked for in the increase in traffic brought thereby to exist-

ing portions of the system. The character of the traffic carried is shown by Table 32. Mr. Osborne.

TABLE 31.

Physical Characteristics.	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Mileage	512	1 117
Of which is Single-track	397	1 117
Double-track	94	....
Quadruple-track	21	....
Ruling gradient	1:300	1: 200
Sharpest curve: Radius, in feet	1 000	500
Actual capital, cost per mile of line	$90 000	$35 500
Of which approximately is due to:		
Preliminary expenses and general charges	2 250	2 350
Rolling stock	21 750	6 725
Track, works and structures	66 000	26 425
Maximum weight of rail	90 lb.	50 lb.
Width of formation	20 ft.	16 ft.
Maximum permitted weight per axle of locomotives, in long tons	22½	10
Minimum seating space per passenger:		
Width	19½ in.	19½ in.
Floor space	3½ sq. ft.	3½ sq. ft.
Capacity	25 cu. ft.	25 cu. ft.
Standard ballast (stone) per foot of track	6 cu. ft	3 cu. ft.
Cost per tie	$1.46	$0.56
Number of locomotives	253	238
Number of passenger vehicles	750	844
Number of freight vehicles	5 498	4 722
Number of stations on the line (including block cabins outside station limits)	152	209
Number of stations interlocked (including block cabins outside station limits)	69	24

TABLE 32.—Passengers and Principal Commodities Carried per Year. (Statistics are not kept Separately for each Section.)

Tons are long tons of 2 240 lb.

Number of passengers	32 975 300
Fibers, raw (mostly loose or not fully pressed), in long tons	1 136 000
Coal, stone, etc	1 107 000
Grain (including flour) and seeds	570 000
Miscellaneous and general merchandise	580 000
Oils, drugs, chemicals, and spices	338 000
Metals and their ores	128 000
Fodder	114 000
Timber, wrought and unwrought	113 000
Salt	106 000
Sugar	84 000
Fibers, spun or woven	80 000
Railway material	72 000

The writer considers that the figures in Tables 32, 33, and 34 prove conclusively that the statement that a narrow-gauge line is more expensive to work than a broad-gauge is not of general application, but needs qualification, depending on the circumstances obtaining in each case. The writer admits the inferiority of a narrow-gauge to broader gauges in one particular only: "Lack of speed." If the speed on the narrow-gauge be sufficiently reduced below that on the broader

Mr. Osborne.

TABLE 33.—Operating Statistics.

Passenger Services.	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Passenger train-miles per year per mile of line	5 139	2 284
Earnings per passenger train per mile (includes mail and express)	$0.99	$0.85
Cost of working a passenger train 1 mile (includes mail and express)....	0.58	0.42
Receipts per passenger mile....	0.0045	0.0043
Expenses per passenger mile....	0.0026	0.0021
Average number of passengers in a train....	184	168
Average weight of a passenger train, in long tons..	264½	174½
Coal per passenger engine per mile, in pounds.....	58	39½
Average speed of passenger trains, including stops, in miles per hour....	21¼	17⅓
Maximum speed of any passenger train, including stops, in miles per hour....	35	25
Freight Services.		
Freight train-miles per year per mile of line.......	3 482	1 583
Earnings per freight train per mile....	$2.10	$1.35
Cost of working a freight train 1 mile....	1.22	0.82½
Receipts per freight ton-mile, in long tons....	0.0101	0.0107
Cost of carrying 1 ton of freight 1 mile, in long tons....	0.0064	0.0065
Average load of freight in a train, in long tons.....	183	121½
Average gross weight of a freight train, in long tons....	551½	352¼
Coal per freight engine per mile, in pounds.......	87½	62¼
Average through speed of a freight train, in miles per hour....	14	12⅔
Ratio of paying freight to total capacity hauled....	34.44%	34.35%

Note.—The writer submits these figures for the purposes of comparison only, and not as the best results obtainable on either a broad-gauge or a meter-gauge.

TABLE 34.—General Statistics.

	5 ft. 6-in. section.	3 ft. 3⅜-in. section.
Cost of maintenance of track, works, and stations, per year per mile of track....	$1 072	$462
Cost of maintenance of track, works, and stations, per year per train-mile....	0.211	0.140
Fuel and all locomotive expenses (including wages and renewals), per train-mile....	0.198	0.174
Car expenses, renewals, cleaning, oiling, etc., per train-mile....	0.076	0.58
All other operating expenses, including station staff, per train-mile....	0.194	0.116
Average cost per long ton of coal at engine shed......	1.85	2.87
General charges: Audit, stores, medical, police, etc., per train-mile.... (not separated) $0.055	....	....
Taxes, legal expenses, claims, payments to other lines, etc., ratio to gross earnings.... (not separated) 5.85%	....	....
Average miles run per engine per day....	*73¼	67¾
Ratio of working expenses to gross earnings....	60%	56%
Ratio of net earnings to total investment....	5⅓%	5⅓%

Note.—The writer submits these figures for purposes of comparison only, and not as the best results obtainable on either a broad-gauge or a meter-gauge.

gauge, all ill effects of curvature, stability, track stresses, maintenance, etc., are reduced to the level of the broader gauge, but it follows, partly as a result of this lack of speed, that increasing traffic will overtop the capacity of a narrow-gauge, single line and demand further capital expenditure on doubling, conversion, or other means to increase its capacity, sooner than it will in the case of a single line of the broader gauge. Mr. Osborne.

In the particular case with reference to which the writer has given statistics, he considers that extensions on the meter-gauge were a mistake, not on account of any inherent defect of the meter-gauge itself, but because of the difficulties arising from the break of gauge. The cost of extending on the broad-gauge would probably not have involved an additional expenditure of more than \$2 500 per mile, against which would have been the saving in rolling stock detained at transhipment junctions and the cost of transhipment terminals, probably sufficient in themselves to wipe out the whole \$2 500 and leave the system to the good, as an operating proposition, by the absence of transhipment costs and the restriction to the development of traffic resulting from delay, loss, and damage to goods at the transhipment junctions.

If the system in question had been isolated, the meter-gauge over the whole system would probably have been as advantageous as the broad-gauge over the whole system; nor is it improbable that a 4 ft. 8½-in. gauge, with standards of construction similar to those of the existing gauges, would have obtained similar results. However, the question of the extensions and their gauge did not arise until the broad-gauge section adjacent to the port had been in operation many years, and the acceptance of the meter-gauge for the whole system was impossible for the following reasons:

1.—The lines which now comprise the system had not been brought under one agency at the time the meter-gauge was commenced, and the then diverse interests rendered impossible a complete system on that gauge.

2.—The broad-gauge was in existence, and its conversion would have been an unjustifiable expense.

3.—The port authorities have always very rightly opposed the entrance of mixed gauges to their docks, jetties, and warehouses.

This case is in some respects similar to the problem the author describes in the Argentine, and the writer holds that the correct solution would have been extension by lines of light construction on the 5 ft. 6-in. gauge, and is so far in agreement with the author's solution for the Argentine alone, namely, gradual conversion to 5 ft. 6-in.

Mr. Osborne.

There is considerable difference, however, between the selection of gauge for new lines and the question of converting existing lines to that gauge. In the first case, the extra expense amounts to little; in the second, it is a very important factor, and there must be considerable saving in the cost of operation to justify it. To the writer the solution appears to be: first, the construction of all future lines to the 5 ft. 6-in. gauge, then the conversion to broad-gauge of the more important narrow-gauge main lines, and afterward the gradual conversion of the less important narrow-gauge branch lines.

The writer, however, is not in accord with the author in his selection of the 4 ft. 8½-in. gauge for all lines in South America other than existing broad-gauge lines in the Argentine, which are to be left as they are.

It is not conceivable to the writer that the efficient, important, and financially strong broad-gauge companies of the Argentine, bound together with mutually friendly interests, will consent to remain as they are, and inactive, while they are being isolated and attacked by the 4 ft. 8½-in. gauge, and thus forced to convert to that gauge. It is much more likely that they will successfully seek to preserve their integrity by extensions opening up new territory, and then, on the Argentine will be thrown all the disablements of a mixed gauge which, in the writer's experience, is much more detrimental to the working of a railroad system as an efficient transportation machine than is the mere question of gauge, provided the same gauge be adhered to throughout. If the possibility (which the writer doubts) of a single gauge for South America be admitted, it should be either 5 ft. 6-in. or 3 ft. 3⅜-in., as the majority of the existing mileage is of those gauges.

The author's selection of a 4 ft. 8½-in. gauge for the whole of South America appears to be a compromise between the existing broad and narrow gauges, and based largely on the assumptions that the results obtained on the 4 ft. 8½-in. gauge of the United States and Canada show their railway systems to be the most efficient in existence, and that this efficiency is due, largely, if not entirely, to their gauge.

The writer, being familiar with the working of two large systems: one a 5 ft. 6-in. system which carries 200 long tons of freight 1 mile at a charge of $1.00 and pays 10% on the investment, the other a meter-gauge system, with only one-sixth the freight traffic density of the former line, which carries 111 long tons of freight 1 mile at a charge of $1.00 and pays 8⅓% on the investment, is inclined to question the first assumption, and, as to the second, without disparagement to those of faith and vision who found in a railway of 4 ft. 8½-in. (the then narrow) gauge an instrument for opening up the interior of the North American continent, or to those whose energy and resource-

fulness have enabled this gauge to cope with present traffic conditions and give the results to-day obtained, the writer's experience is that, under conditions now obtaining in this portion of North America, the 4 ft. 8½-in. gauge is a mistake, and that if the gauge were 5 ft. 6-in. the country would now be better served, and for reasons similar to those given by the author in casting the narrow gauge in favor of a broader one. Mr. Osborne.

The forces which have impelled the 4 ft. 8½-in. gauge in North America to economize by increasing the train load and so allowing revenue-ton-miles to increase while reducing revenue-train-miles are world wide, and where the 5 ft. 6-in., meter and other gauges have a live management, they affect them in the same way as they have affected the North American 4 ft. 8½-in. gauge, and South America will gain nothing that she would not otherwise receive along these lines by tearing up her existing 5 ft. 6-in. and 3 ft. 3⅜-in. gauges to replace them by 4 ft. 8½-in.

If the conditions and density of traffic in the area under discussion (Bahia Blanca to north of Rio) are ever likely to approach the conditions and density of traffic of the railways of the United States and Canada, then the extension of the Argentine 5 ft. 6-in. is undoubtedly the gauge that should be made universal.

The writer doubts whether such will ever be the case. There may be considerable similarity between conditions in the southern part of the United States and the southern part of this area, but at their northern extremities one is in the Tropical and the other is in the North Temperate Zone. So far as the writer's experience goes, the inhabitants of a tropical climate do not develop the vigor and capacity for intense effort that lead one to suppose they will ever develop a commerce sufficient to require and justify the railway facilities given in Northern Europe and the northern part of North America. The development of the meter-gauge in the tropical and the broad-gauge in the temperate portion of the area under consideration is not improbably an adjustment to the economic needs of each area, and the writer is inclined to look for the economic and economical development of these areas and their railways along the existing majority gauges, and would endeavor to consolidate the northern lines on the meter- and the southern lines, with the Argentine lines, on the broad-gauge, the dividing line between the gauges, in the absence of political and military considerations, being the traffic-shed between areas tributary to the ports of Buenos Aires and Rio, across which but little traffic would pass.

The writer's experience is that all gauges are good and capable of efficient operation where the standard to which the line is constructed is suitable to the character of the traffic to be moved, but that mixed

Mr. Osborne. gauges and breaks of gauge are an unmitigated evil. In the situation disclosed by the author, the needs would be met by preserving both the 5 ft. 6-in. and 3 ft. $3\frac{3}{8}$-in. gauges, which comprise about 80% of the existing railway mileage, as each gauge could be confined to a definite area where the disabilities which arise from a break of gauge on a line of large through traffic would not occur.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE POSSIBILITIES IN BRIDGE CONSTRUCTION BY THE USE OF HIGH-ALLOY STEELS.

Discussion.*

By Messrs. M. Séjourné and George L. Norris.

M. Séjourné, Esq.† (by letter).—The writer takes great interest in the problem of the use of steels of high elastic limit for the construction of bridges, and would second the author's efforts to obtain in France—at least for large works—the use of a different kind of steel from that which is now specified, and having the following characteristics:‡ Mr. Séjourné.

$R = 42$ kg. per sq. mm.

$E = 24$ kg. per sq. mm.

$A = 25\%$ in a standard length, $L = \sqrt{66.67\ S}$

In France it is not usual to construct such great bridges as in America; consequently, the use of steels of high resistance is not as necessary, and the French steel works hesitate to supply themselves with tools for producing in great quantities steels which they are not sure of selling.

The writer is at present engaged in designing a large metal bridge over the Durance, the longest span being 121 m. (397 ft.). On account of the importance of this work, one of the largest steel works in France was requested to quote prices for furnishing carbon steel having the following characteristics:

$R > 50$ kg. $\quad E > 27$ kg. $\quad A = 20$ per cent.

The steel works replied that it would agree to these specifications, provided that an increase in price of 1 Franc per 100 kg. more than for ordinary 42-kg. steel was allowed, but without impact test.

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* Continued from August, 1914, *Proceedings*.

† Ingénieur en Chef du Service, Chemins de Fer de Paris à Lyon et à la Méditerranée.

‡ It will be noted that these characteristics differ from those given to the author and reproduced by him in his article in *Le Génie Civil*, No. 15, August 7th, 1909.

Mr. Séjourné. In using steels of high elastic limit it seems to the writer to be indispensable to have all the necessary guaranties with reference to its brittleness, and to have recourse, for that, to methods even more stringent than those now used for 42-kg. steel.*

Although French steel works allow tests for brittleness for high-priced special steels, they are generally opposed to them for steels of ordinary make designed for bridges and metal frameworks. These conditions will perhaps be bettered, thanks to developments in the manufacture of steel due to the electric furnace, which produces a material that is not brittle because it is pure. Unfortunately, its price is still too high.

The writer commends the author's endeavors to discover a constituent alloy of great tensile strength, high elastic limit, and of reasonable price. The problem is doubtless difficult to solve, but it is not beyond the science and ingenuity of American engineers, and it is hoped that the author's researches and tests may result in the desired metal.

Mr. Norris. George L. Norris, Esq.† (by letter).—The author has shown the great advantages to be derived in bridge construction, especially for long spans, by the use of steel with high elastic limit. To obtain the high elastic limits which he mentions—60 000 to 100 000 lb. per sq. in.—it will be necessary to use alloy steels.

With alloy steel it is entirely feasible to obtain elastic limits of from 90 000 to 100 000 lb. by heat treatment, and even also in the condition as rolled, but this, of necessity, would increase considerably the difficulties of shop manipulation. In the condition as rolled, the alloy steels would naturally be too variable in hardness for safe use.

In the case of nickel steel, in order to attain an elastic limit of 60 000 lb. or more, it is necessary to increase the percentage of carbon to undesirable limits.

To produce steel having an elastic limit approaching 100 000 lb., and workable under ordinary shop manipulations, the writer believes that recourse will be had to the use of vanadium alone, or with some other metal such as chromium in the steel. Vanadium is undoubtedly the element which, together with carbon, acts with the greatest intensity in improving alloys of iron, that is, in very small percentages. The quantity of vanadium generally alloyed with steel, excepting tool steels, is about 0.15 or 0.20 per cent. The price of vanadium has been reduced 60% within the past 3 years, and it is now low enough to

* In reference to the information given to the author, and reproduced by him in his article in *Le Génie Civil* (August 7th, 1909), it is said that the specifications of the French railroads "do not require impact tests, which is greatly to be regretted." This is true, but the writer does not make use of these specifications in his office, and for 12 years he has required such tests on all rolled steel.

† Metallurgical Engr., American Vanadium Co., Pittsburgh, Pa.

bring it within consideration for use in steel for eye-bars and other parts of long-span bridges. Mr. Norris.

Vanadium has generally been used in combination with chromium, although when used with nickel or nickel and chromium it gives greatly increased physical properties; these latter combinations are naturally more expensive. Added to simple carbon steel, especially if the percentage of manganese is more than 0.60%, it gives an increase of about 40% in the elastic limit and about 20% in the tensile strength, with practically the same or even greater ductility.

What is known as Type "A" chrome vanadium steel will fulfill the requirements of high elastic limit and be workable under shop manipulations. The cost of this steel would be only slightly in excess of that of 3¼% nickel steel. Its chemical range would be:

Carbon	0.17 to 0.27	per cent.
Manganese	0.40 to 0.60	" "
Chromium	0.60 to 0.90	" "
Phosphorus, not more than	0.05	" "
Sulphur, not more than	0.05	" "
Vanadium, not less than	0.15	" "

As rolled, in plates, shapes, and bars, this steel would have approximately the following properties:

Elastic limit	Tensile strength.	Elongation in 8 in.	Reduction of area.
60 000 to 80 000 lb.	85 000 to 110 000 lb.	12% to 20%	45% to 60%

By the simple operation of heating to 1 100 or 1 150° Fahr., any irregularities in hardness and strength, due to variation in rolling temperatures or uneven cooling, can be removed without appreciably decreasing the elastic limit and tensile strength, and, at the same time, increasing the ductility. This operation would not be an expensive one, and experience might show that it would be unnecessary for this grade of steel.

Type "A" chrome vanadium steel can be sheared, punched, reamed, bent, etc., without any considerable increase of shop manipulations.

Much higher elastic limit and tensile strength can be obtained from this steel by heat treatment, quenching and tempering, but this would increase the shop manipulations very materially, necessitating drilling instead of punching and reaming. It would also add materially to the cost. Any bends would have to be made before heat treatment.

Rather than heat-treat steel of this type, it would be preferable to attain elastic limits of 80 000 to 100 000 lb. by increasing the carbon percentage, and normalize the steel by heating to 1 100 or 1 150° Fahr.

Mr. Norris.

The range in chemical composition for this grade would be:

Carbon	0.30 to 0.40 per cent.
Manganese	0.40 to 0.60 " "
Chromium	0.60 to 0.90 " "
Phosphorus, not more than	0.05 " "
Sulphur, not more than	0.05 " "
Vanadium, not less than	0.15 " "

Plates $\frac{1}{8}$ in. thick, in the upper range of this composition, have shown the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 2 in.	Reduction of area.
125 000 lb.	145 000 lb.	18%	58%

These plates could be punched and sheared as rolled, but it would be advisable no doubt to normalize, as before described, and also to drill rather than to punch the holes.

Plates $\frac{5}{8}$ in. thick, at the low range of this composition, have shown the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 2 in.	Reduction of area.
83 000 lb.	116 000 lb.	20%	45%

Normalized, this grade of steel should give, in plates and shapes, the following physical properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
75 000 to 100 000 lb.	100 000 to 125 000 lb.	More than 12%	More than 50%

The cost of these chrome vanadium steels would be about 3 cents per lb. more than for ordinary carbon steel.

The writer believes that a simple carbon vanadium steel would prove commercially more attractive than the vanadium steels containing chromium or nickel or both. It is possible to obtain a very material increase in elastic limit by the addition of vanadium to a simple carbon steel, and such steels can be manipulated almost, if not quite, as readily as ordinary carbon steel.

Tests from $1\frac{1}{16}$-in., round, rolled bars of acid open-hearth casting steel of the following composition give a very good idea of what can be expected from this type of steel and how it compares with simple carbon acid open-hearth casting steel:

Carbon	0.26 to 0.27 per cent.
Manganese	0.60 to 0.64 " "
Silicon	0.25 to 0.25 " "
Vanadium	0.21 to 0.00 " "
Phosphorus, less than	0.05 to 0.05 " "
Sulphur, less than	0.05 to 0.05 " "

Mr. Norris.

Elastic limit	70 000	to	56 000	lb.
Tensile strength	88 000	to	72 000	"
Elongation in 2 in.	28.5	to	34	per cent.
Reduction of area	57.5	to	58	" "

From these and other tests, it would seem feasible to specify as follows, for this type of steel:

Carbon	0.20	to	0.30	per cent.
Manganese	0.60	to	0.80	" "
Phosphorus, not more than			0.05	" "
Sulphur, not more than			0.05	" "
Vanadium, not less than			0.15	" "

Elastic limit	50 000	to	70 000	lb.
Tensile strength	80 000	to	100 000	"
Elongation in 2 in., more than			20	per cent.
Reduction of area, more than			45	" "

The additional cost of vanadium steel of this grade over that of ordinary carbon steel should not be more than 1 cent per lb., and very likely would be less.

For built-up members of vanadium steel, the writer believes that vanadium steel rivets should be used, in order to utilize more fully than would be otherwise possible the high physical qualities of the vanadium steel shapes and plates. The rivets could be either of chrome vanadium steel or simple carbon vanadium steel. The composition for chrome vanadium steel rivets should be:

Carbon	0.15	to	0.20	per cent.
Manganese	0.30	to	0.50	" "
Chromium	0.40	to	0.60	" "
Phosphorus, not more than			0.05	" "
Sulphur, not more than			0.05	" "
Vanadium, not less than			0.15	" "

This steel, as rolled in rounds, would have the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
50 000 to 65 000 lb.	70 000 to 90 000 lb.	More than 18%	More than 50%

Single and double shear tests of rivets of steel of this type show:

For single shear.....20 per cent.
For double shear.....30 " "

greater than for ordinary rivets with a tensile strength of 55 000 lb.

Mr. Norris.

The simple carbon vanadium steel should have a chemical range of:

Carbon	0.15 to 0.20 per cent.
Manganese	0.60 to 0.80 " "
Phosphorus, not more than	0.05 " "
Sulphur, not more than	0.05 " "
Vanadium, not less than	0.15 " "

This steel, as rolled in rounds, would have the following properties:

Elastic limit.	Tensile strength.	Elongation in 8 in.	Reduction of area.
40 000 to 55 000 lb.	65 000 to 85 000 lb.	More than 18%	More than 50%

There should be no difficulty in driving rivets of either of these types of steel.

In the case of eye-bars, possibly the conditions are more favorable for the use of alloy steels than for built-up members. They can be more readily and advantageously heat-treated to develop high elastic limits.

In 1909 a number of tests were made of full-sized eye-bars, heat-treated, of chrome vanadium and chrome nickel vanadium steel. These eye-bars were made from bars 14 by 2 in., had 34-in. heads with 12-in. pin-holes, and were 25½ ft. long over the pin-holes.

The chrome vanadium steel bars experimented with were too high in carbon, and the results obtained for elongation in 20 ft. were not quite as good as in the case of the bars from the chrome nickel vanadium steel. Tests from this latter steel gave results ranging as follows, depending on the drawback or annealing temperature after quenching:

	No. 1.	No. 2.
Elastic limit	63 280 lb.	80 480 lb.
Tensile strength	93 500 "	99 800 "
Elongation, 12 in.	35 per cent.	32.5 per cent.
Elongation, 20 ft.	14.2 " "	7.9 " "
Reduction of area	50.8 " "	52.3 " "

Tests on 2 by ½-in. specimens turned up from the disks cut out in machining the eyes check the elastic limit and the tensile strength obtained from the full-sized bar very well. A test from the eye disk of Bar No. 2 showed:

Elastic limit	83 040 lb.
Tensile strength	94 140 "
Elongation, 2 in.	25 per cent.
Reduction of area	71.9 " "

The chemical composition of this steel was approximately: Mr. Norris.

Carbon	0.25	per cent.
Chromium	0.90	" "
Nickel	1.20	" "
Vanadium	0.17	" "

Based on these tests, it is perfectly feasible to specify as follows for heat-treated eye-bars of chrome vanadium or chrome nickel vanadium steel:

Elastic limit	65 000 to 80 000	lb.
Tensile strength	85 000 to 105 000	"
Elongation in 2 in.	20	per cent.
Reduction of area	50	" "

The cost of heat treatment for eye-bars would probably be from ¼ to ⅜ cents per lb. The cost per pound of the chrome vanadium or chrome nickel vanadium steel would be about 3 cents more than that for carbon steel.

The writer believes that eye-bars made from simple carbon vanadium steel of the following composition, either heat-treated, quenched and annealed, or normalized, heating to about 1 100 or 1 150° Fahr., after the heads have been forged, will give elastic limits approaching those of the more expensive chrome or chrome nickel vanadium steels:

Heat-treated or normalized:

Chemical composition.

Carbon	0.30 to 0.40	per cent.
Manganese	0.60 to 0.80	" "
Phosphorus, not more than	0.05	" "
Sulphur, " " "	0.05	" "
Vanadium, not less than	0.15	" "

Physical properties.

Elastic limit	60 000 to 75 000	lb.
Tensile strength	85 000 to 100 000	"
Elongation in 2 in., more than	18	per cent.
Reduction of area, more than	45	" "

The writer has given considerable attention to simple carbon vanadium steel containing from 0.60 to 0.80% of manganese. This steel will be the commercial or every-day vanadium steel of the immediate future. It is much cheaper than the vanadium steels containing chrome or nickel. It is at least 40% better than simple carbon steel of otherwise the same composition, and is bound to be used extensively in the near future for rails, general and locomotive forgings, and special structural purposes. It presents no especial difficulties in manufacture over simple steel.

Mr. Norris. The quantity of vanadium which remains in the steel is about 80% of that added. It is very evenly distributed. No instances of segregation are known, and it has a strong influence in overcoming the segregation of other elements, particularly carbon.

Vanadium will readily alloy with nickel, and better results can be obtained from a 2% nickel vanadium steel than from a 3½% straight nickel steel of the same carbon content.

There would be absolutely no advantage gained in the use of titanium with vanadium. Vanadium is an alloying metal and is used as such, not as a scavenger or deoxidizer. It is only the vanadium which alloys with the steel that can be considered as influencing the quality of the latter. Titanium has a little merit as a deoxidizer over silicon or aluminum, and its action is apparently a surface reaction. It is a question whether the observable reaction, when evident, is not with the highly oxidizable basic slag with which it comes in contact, or even with the atmosphere.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE DETERMINATION OF SAFE YIELD OF UNDERGROUND RESERVOIRS OF THE CLOSED-BASIN TYPE.

Discussion.*

By Messrs. O. E. Meinzer and Kenneth Allen.

O. E. Meinzer, Esq.† (by letter).—Mr. Lee's investigation in Owens Valley is a valuable contribution to the study of ground-water. The demand for irrigation supplies and the increasing availability of ground-water because of improved irrigation and cultural methods and decreased pumping costs have created a need for information as to the magnitude of ground-water supplies, the question being primarily not as to the quantity stored in the earth but as to the annual recharge or safe yield. Any contribution to the methods for estimating the annual supplies of ground-water, therefore, is especially valuable at this time. Four principal methods or groups of methods, which may be called the percolation, underflow, water-level, and evaporation methods, have been used. The first consists in estimating the quantity of water that percolates into an underground reservoir from streams or other surface sources; the second in measuring the flow of ground-water at selected cross-sections, being similar in principle to the gauging of surface streams; the third in observing fluctuations in the water-table, which represent filling or emptying of the underground reservoir; and the fourth in measuring the discharge of ground-water through evaporation from soil and plants. All these methods are laborious and difficult to apply, and none of them can be expected to produce precise results, but they are valuable, nevertheless, because they give some tangible basis for estimating ground-water supplies. Mr. Meinzer.

* Continued from August, 1914, *Proceedings*.

† In Charge, Ground-Water Div., Water Resources Branch, U. S. Geological Survey.

Mr. Meinzer. In the Owens Valley investigation, Mr. Lee used both percolation and evaporation methods, his distinctive contribution consisting in developing the latter method and placing it on a quantitative basis.

The evaporation method gives promise of extensive applicability because a large number of the ground-water reservoirs that will be developed for irrigation, discharge wholly or chiefly by evaporation. Débris-filled basins are the most characteristic features of the United States west of the Rocky Mountains. They are of two types: Those which discharge ground-water through springs and evaporation areas, as described by Mr. Lee, and those which do not. In the geologic literature dealing with these basins this fundamental distinction is generally ignored, and the characteristics of the evaporation areas are commonly accounted for by the wholly inadequate explanation of evaporation of surface waters. The process of ground-water evaporation, however, has been clearly stated by F. H. Newell,* M. Am. Soc. C. E., in one of the earliest papers on water resources published by the United States Geological Survey, and in recent ground-water investigations the significance of evaporation areas has come to be clearly recognized. Mr. Lee has furnished experimental data showing that these areas are quantitatively important in discharging ground-water.

Work in numerous débris-filled basins has shown that it is entirely feasible to ascertain from surface indications whether or not a basin is discharging ground-water by evaporation. The evaporation areas in some of the basins have been mapped with nearly the same accuracy that is possible in mapping geologic formations. The three criteria, all of which are suggested in the paper, are (1) moisture of the soil; (2) soluble salts at the surface; and (3) native plants that feed on ground-water. Experience is necessary, of course, for a proper application of these criteria, but they are trustworthy when rightly used. Moreover, they can be tested at any time by making a shallow boring, for ground-water evaporation takes place only where the water-table is near the surface. In a recent ground-water survey of the Big Smoky Valley, Nevada, the relations between the vegetation zones and the depth to the water-table were found to be very similar to those stated by Mr. Lee (page 867),† but the plant species that serve as indicators of ground-water are not the same in different parts of the West. The mapping of evaporation areas is important not only because these areas make possible estimates of the annual supplies, but also because they reveal the base level of the ground-water surface, and thus make possible a forecast of the depth to water in other parts of the basins in which they occur.

Mr. Lee's first conclusion (page 818)† is open to criticism in being too general. On the basis of his investigation in the Owens Valley

* "Water Supply for Irrigation," U. S. Geol. Survey, 13th Annual Rept., Pt. 3, p. 29, 1893

† *Proceedings*, Am. Soc. C. E., for April, 1914.

he concludes that the underground reservoirs of California and the Southwest are water-tight rock basins. Many of the débris-filled basins of the Southwest, however, even those comparatively well enclosed by mountains, have no ground-water discharge through springs, evaporation, or transpiration, and it must be assumed that the supplies which they undoubtedly receive are disposed of entirely through rock absorption or underground channels of escape. Some of the reservoirs which discharge water by springs, soil evaporation, and transpiration, no doubt also suffer losses through rock absorption or leakage. In this respect each basin forms a separate problem, the amount of underground loss depending on the stratigraphy and structure as well as the topography. It should be noted that such loss, if heavy, will make the estimates obtained by the percolation method too large and may make those obtained by the evaporation method too small. One of the sources of the public supply of Goldfield, Nev., has consisted of wells in a closed débris-filled basin which has no discharge by evaporation. Such a basin will yield some water even though it has no natural discharge through springs, evaporation, and transpiration, but its yield will be less than the quantity that percolates from surface sources to the water-table. Mr. Meinzer.

The writer was disappointed in not finding in Mr. Lee's paper a more definite discussion of the probable percentage of accuracy of his results, as such a discussion would have added greatly to the value of the paper. Mr. Lee's analyses both of recharge and of loss are excellent, but in order to reach quantitative conclusions he was obliged to make numerous assumptions in respect to both. Although these assumptions were, the writer believes, made carefully and with good judgment, they must have introduced considerable errors into the computations. As assumptions enter into both estimates, neither one can be regarded as a check on the other. The fact that the two estimates are of the same magnitude justifies added confidence in the general results, but the fact that they differ by less than 4% does not indicate, of course, that the percentage of error is within 4%, and can hardly be considered a confirmation of the reliability of the data and the correctness of the assumptions in either computation (page 881)*.

The four sources of percolation are given as (1) direct precipitation; (2) stream flow; (3) irrigation; and (4) flood-water in the valley floor. It is assumed that the high mountain areas shed all precipitation except that which is lost by evaporation; that on the more elevated parts of the intermediate mountain slope 75% of the precipitation, and on the lower parts 70, 65, and 60%, respectively, join the underground supply; that the contributions to the underground supply on the outwash slopes range, according to zones, from 0 to 60%, making an average of 16%; and that the contribution on the valley floor is 4 sec-ft. All these assumptions are based on care-

*Proceedings, Am. Soc. C. E., for April, 1914.

Mr. Meinzer. ful, general observations, but are of course only approximations and liable to large errors. Moreover, assumptions are involved as to the areas belonging to the various zones and the amount of precipitation in each (page 875)*. Several assumptions are also made in the estimate of recharge from stream channels, in the conclusion that 50% of the irrigation water is added to the underground supply, and in the estimate of the contributions made by the flood-waters.

The writer agrees with Mr. Lee that the discharge from the underground reservoir can be determined more accurately than the percolation into it, but the methods of estimating discharge also involve a number of elements of uncertainty. If the writer understands rightly the author's discussion of the discharge from irrigated land, the 50%, or 18 sec-ft., of irrigation water discharged by evaporation and transpiration (page 872)* is the portion that was not added to the underground supply (page 881)* and, therefore, should not be included with the discharge from this supply.

The largest element in ground-water discharge is that of evaporation and transpiration from uncultivated land. It is on this question that Mr. Lee has made his most important contribution, and that part of his paper dealing with this phase of the subject deserves, therefore, the most critical consideration. Some of the factors that enter into this estimate were determined by experiment and others by field survey. Experimental errors were involved in the difference between natural and artificially packed soils, in the difference between the vegetation in Nature and in soil tanks (pages 855, 856, 861, and 862),* in uncertainties as to the actual water-level in the tanks (pages 852 and 853),* and in the unavoidable fluctuations in the water-levels in each tank (Tables 9 to 14, inclusive). These experimental errors are represented in part only in the diagrams in Fig. 10. In the summer diagram, which is the more important one, the results from Tanks Nos. 5, 6, and 7 fall nearly on the curve used by Mr. Lee in his calculations, the result from Tank No. 4 being more than 20% too low, that of Tank No. 3 more than 20% too high, and that of Tank No. 2 nearly 40% too low. It should be noted, however, that although these results involve large experimental errors, they corroborate each other in a general way and are of great value in furnishing reliable data on the general magnitude of one of the most important processes in ground-water circulation.

In applying the experimental data to the basin under investigation, inaccuracies were involved in the sizes of the areas having specified depths to the water-table, in the fluctuations of the water-table, in difference in the range and rate of capillary rise due to difference in the character of the soil, and in difference in the density of vegetation

* *Proceedings*, Am. Soc. C. E., April, 1914.

Mr. Meinzer.

and kinds of plants that draw water from the underground reservoir. An error was also involved in the fact that observations covering only 1, 2, or 3 years did not give average evaporation conditions, just as precipitation and stream-gauging data for a period of the same length do not afford reliable averages. With 142 observation wells on the valley floor (page 866),* the error as to the water-table cannot have been large, yet the rate of discharge varies so greatly with small changes in depth of the water-table that even slight inaccuracies in determining one produce appreciable errors in calculating the other. Although the valley floor in that part of Owens Valley investigated by Mr. Lee has relatively uniform conditions, there is luxuriant salt grass in some parts and an entire absence of vegetation in others (page 828),* whereas the experiments did not cover these different conditions. Moreover, no account was taken of the zone having depths to the water-table between 8 and 12 ft., although the greasewood and rabbit brush in this zone probably draw water from the underground reservoir (page 867).*

The writer agrees with Mr. Lee that the evaporation method is the most feasible one for estimating ground-water recharge in many of the débris-filled basins, especially where there are as yet few developments, but its application is far from being a simple matter. The data which he obtained in Owens Valley have value in making rough estimates of annual recharge in valleys in which the evaporation areas are mapped and reliable observations are made as to the character of the soil and vegetation and the distance to the water-table in the different zones of such areas, but to assure any considerable degree of accuracy for most valleys it will be necessary not only to sink a large number of observation wells and to keep them under observation for one or more years, as was done in Owens Valley, but also to obtain a great deal more information as to the rate of discharge under various conditions of soil and vegetation. The conditions in the evaporation areas of most valleys are far from uniform, the soil ranging from dense clay to coarse sand or gravel, and the vegetation embracing a number of diverse species and being entirely absent over large tracts.

The writer wishes to urge the importance of further investigations along the lines suggested. The lowest parts of many of the closed basins are underlaid by clay cores destitute of vegetation but surrounded by zones of less dense soil in which evaporation and transpiration are active. The dissemination of the soluble salts instead of their concentration at the surface and other conditions lead him to believe that on these clay cores the ground-water discharge is sluggish, but definite tests are needed as to the quantity of discharge and its relation to the distribution of the soluble salts. Among the common native plants (besides the salt grasses) which apparently discharge ground-water, are samphire (*Spirostachys occidentalis*), iodine weed (*Suaeda*),

**Proceedings*, Am. Soc. C. E., April, 1914.

Mr. Meinzer. alkaline sacaton (*Sporobolus airoides*), certain species of salt bush (*Atriplex*), big greasewood (*Sarcobatus vermiculatus*), rabbit brush (*Chrysothamnus graveolens*), buffalo berry bush (*Shepherdia*), and mesquite (*Prosopis*). Mesquite does not thrive in the shallow-water areas where the soil is dense and alkaline, but is often dominant in a zone of moderate depth to water surrounding a shallow-water area. It is important to know whether the mesquite actually feeds on ground-water, and if so, from what depth and at what rate it is able to lift this water to the surface.

Mr. Lee's final conclusions (page 887)* sum up admirably the esssential factors in the problem of safe yield. As it is not generally practicable to draw any large part of the ground-water of one segment of a valley to another, a proper distribution of wells is necessary in order to reduce the residual losses to the lowest possible quantity. Overdraft with serious lowering of the water-table may occur in certain segments while evaporation of ground-water contributed by other segments is still in progress on the lowest lands. Such loss can be prevented only by increased withdrawals in the undeveloped segments, and if these segments have little or no land that can be profitably irrigated, the loss may be unavoidable.

Mr. Allen. KENNETH ALLEN, M. AM. SOC. C. E. (by letter).—Underground water supplies, as well as surface supplies, depend on the rainfall, the catchment area, and the available storage, with its accompanying losses by overflow, leakage, and evaporation; but the problem for the engineer is more difficult in the case of underground supplies, as the true limits of the catchment area, the storage capacity, and the probable amount of the losses mentioned can only be determined approximately after a pretty thorough examination of the sub-surface conditions, that is, the configuration of the permeable strata, their impermeable confines, and their physical characteristics—permeability, etc. In the majority of cases much of this information is inaccessible, and more or less dependence is placed on such collateral evidence as the yield of neighboring wells and the results obtained by sinking test wells.

The limitations imposed by enclosing impervious formations and by sub-surface evaporation are clearly and interestingly presented by the author, and their importance is shown under the conditions discussed. In most well developments, however, evaporation is of minor significance. This is not only on account of greater humidity, but because most supplies percolate through an unenclosed stratum for perhaps many miles and because evaporation from this stratum is usually prevented by the superposition of one or more impervious strata.

* *Proceedings*, Am. Soc. C. E., for April, 1914.

In practice the available yield is subject to further limitations. The water-bearing stratum may consist of an impervious rock containing crevices or fissures due to movements in the earth's crust or water-courses caused by the solvent action of the water itself. The latter are of frequent occurrence in limestone and chalk formations and the former in sandstones and the denser igneous rocks. Fissures occur more frequently near the surface than at great depths, and, as the cost of drilling increases with the depth, deep borings in search of water-bearing fissures are not often profitable. The same money can be spent to better advantage in making several test borings at moderate depth. Although the results of such borings are uncertain, supplies, when obtained from fissures, are often abundant and the wells free from the clogging so often experienced with sand. The yield of a well in sand is directly limited by the porosity of the latter and the available head. In fine sands there is a large loss of head due to percolation near the strainer in order to maintain the necessary flow. This difficulty may be overcome by increasing the length of the strainer if this does not exceed the thickness of the water-bearing stratum, but this is at the expense of a greater first cost as well as an increase in lift due to the fineness of the material. In such cases the loss of head is often reduced by removing the sand about the strainer and filling the pocket with gravel, or else by substituting a larger number of driven well points from 2 to 3 in. in diameter for the large (6 to 12-in.) casing and strainer. Mr. Allen.

In those cases where the water occurs in a stratum of fine sand of small thickness, the difficulty is further increased. The writer tested a stratum of this kind several years ago with the view of developing a supply of some 5 000 000 gal. daily for a Southern city. Water was found in apparent abundance throughout a large area at a moderate depth, the land was low, covered with forest, not far from a stream of considerable size, and on it were several large springs. On sinking numerous test wells on a line about 4 miles long, however, it was shown that the water-bearing stratum, besides being of a fine sand, was so thin that the cost of developing the desired supply would be prohibitive.

Water supplies found in deep deposits of sand or drift, especially those derived from distant and extensive catchment areas, such as exist south of the great terminal moraine on the south side of Long Island and on the Atlantic Coastal plain from Sandy Hook to Florida, are most favorable for development. Well-known examples are the 1 400-ft. Ponce de Leon well at St. Augustine, said to furnish 10 000 000 gal. daily, and the 1 970-ft. well at Charleston furnishing 1 250 000 gal. daily. On the other hand, a well was bored in 1900-01 at Atlantic City, N. J., to a depth of 2 306 ft. (below the floor of Young's Pier), and although a good supply was found between depths of 780 and 860

Mr. Allen. ft., in a stratum of sand tapped by numerous wells in the vicinity, the supply sought at the greater depth was not realized, and the well, then the deepest but one along the Atlantic Coast, was abandoned.

About 5 miles inland, the Water Department had a small collecting basin near its pumping station (Fig. 15), in which were driven a number of 4-in. tubes tapping a water-bearing stratum 24 ft. below. Near the basin was a 10-in. well (No. 3) extending to a depth of about 100 ft. to another water-bearing stratum which it was proposed to develop as an additional supply to the extent of 3 000 000 or 4 000 000 gal. daily. The capacity of this well was first tested by erecting over it an old 1 000 000-gal. Worthington pump, which the Department had, and connecting it with a steam line to the boiler plant at the station. During a 6-hour test with a 19-in. vacuum on the suction,

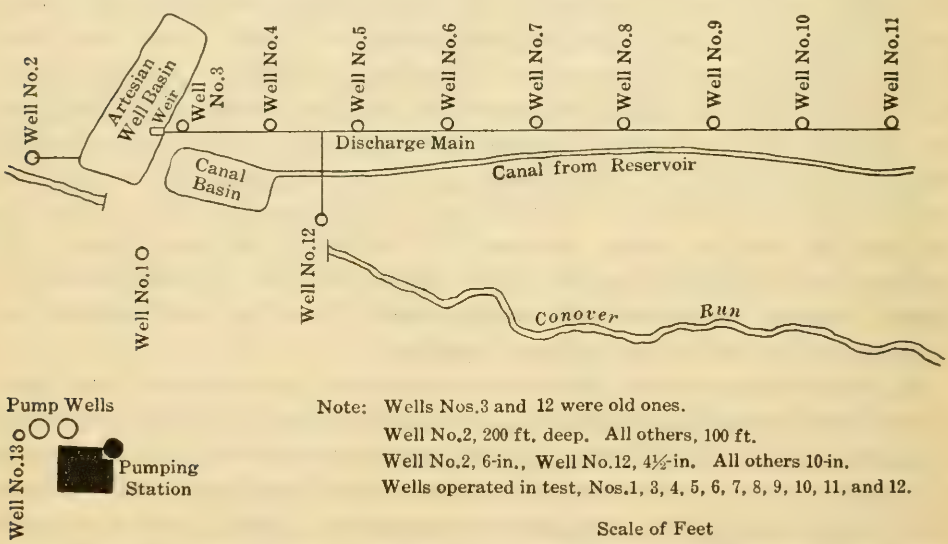

FIG. 15.

the delivery was practically uniform at a rate of 840 000 gal. per day, and as no downward flow was observed in the small tubes in the basin, it was concluded that there was no connection between these two strata. On the strength of this test, ten 10-in. wells, 125 ft. apart, were sunk to a depth of about 100 ft. and a 6-in. well (No. 2) to a still deeper stratum, 200 ft. below the surface. Besides these there were two old wells—the 10-in. well tested with the steam pump and a 4½-in. well (No. 12), both of which were carried to the 100-ft. stratum. The deeper 6-in. well flowed freely at a rate of 66 000 gal. per day and, by use of the air lift, at a rate of 400 000 gal. per day, or about 280 gal. per min. The 10-in. wells on short separate tests produced from 150 to 400 gal. per min. Ten of the 10-in. wells (No. 1 and Nos. 3-11) and the 4½-in. well (No. 12) were then connected with

the compressor and given a 6-hour test. These wells together delivered 846 879 gal., or at a rate of 3 705 492 gal. per day. This was an average of 336 866 gal. per well, or 60% less than the delivery from Well No. 3 when operated alone. Moreover, the average lift in this well during the test was 24.56 ft., though the vacuum gauge on the suction during the earlier separate test indicated a lift of 21 ft. to about the same level. Assuming this well to have produced one-eleventh of the total quantity during the combined test, the effect of interference from the other wells, while operating all eleven at this rate, was to increase the lift by about 3.6 ft. while reducing the output by 60 per cent. Mr. Allen.

The yield under conditions found along the Coastal Plain region bears little relation to that from underground reservoirs of the closed-basin type described by the author, but somewhat similar conditions are met in the basin of the prehistoric Lake Passaic of Northern New Jersey. Wells in the outlet of this lake, now filled with glacial drift, furnish the water supply for East Orange. When first sunk, several of these 6-in. and 8-in. wells had a natural flow of from 400 to 500 gal. per min.

Above this outlet water-bearing sandstones underlie the ancient lake, but percolation to these beds is cut off on the northwest by the trap dikes forming the Orange or Wachung Mountains and on the southeast by the Palisades of the Hudson. Assuming a general southerly flow, percolation from a distance, therefore, is limited to that from the northeast, the limit in this direction, it is believed, being unknown.

These sandstones, nevertheless, furnish a large number of excellent well supplies in the vicinity of Newark and Paterson, chiefly by means of their numerous fissures.*

Data concerning a large number of these wells, collected by the writer a few years ago, indicate a great variation in the yield. They are commonly from 100 to 500 ft. in depth, and 6 or 8 in. in diameter, and generally furnish from 20 to 200 gal. per min. each, although failures are not infrequent, and several produce as much as 500 gal. per min.

* Report, State Geologist of New Jersey, 1903, p. 79; also Water Supply and Irrigation Papers, U. S. Geological Survey, No. 114, p. 96.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

REINFORCED CONCRETE DOCKS: FOREIGN AND AMERICAN STRUCTURES. FAILURES, COSTS, AND GENERAL CONSIDERATIONS.

Discussion.*

By E. G. Walker, Assoc. M. Am. Soc. C. E.

Mr. Walker.

E. G. Walker, Assoc. M. Am. Soc. C. E. (by letter).—At first thought it might appear remarkable that so little progress relatively has been made in the application of reinforced concrete to marine works in the United States; and in certain places in his paper, the author seems to attribute the present state of affairs to a conservatism of ideas on the subject utterly opposed to the general conception of the American progressive spirit. The writer is inclined to differ from this deduction, because he believes that in the United States as elsewhere the form of construction adopted for marine works is the result of economical evolution. A paper† on the piers of New York Harbor, recently presented before the Society, is an admirable illustration of this point, showing as it does in a very conclusive manner that special conditions evolve special types. A consideration of this paper leaves little doubt that the engineers of the Department of Docks and Ferries have arrived, by the judicious use of timber in some parts and reinforced concrete in others, at a type of pier of maximum economy under the conditions prevailing in New York Harbor. These conditions, however, are not comparable with those obtaining in certain teredo-infested waters on the coast of Great Britain, for example, where pitch pine piles will be dangerously eaten away in a few years

* Continued from August, 1914, *Proceedings*.

† "Modern Pier Construction in New York Harbor," by Charles W. Staniford, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 503.

Mr. Walker. and where even Australian hardwood piles have to be renewed at comparatively short intervals. Although there appear to have been a number of unfortunate experiences in the United States, there is no question that concrete and reinforced concrete can be used successfully for marine works, because engineers have had ample experience in the use of both materials; but there are many locations where it may be more economical to use timber, in spite of its shorter life and other disadvantages. At the present time there is a large quay under construction on the River Tyne, in England, forming the largest item in a $1 000 000 scheme, which will be built of timber throughout. In North America, where timber is so much more easily obtainable than in the United Kingdom, the writer thinks that it will be a long time before it is completely ousted by reinforced concrete.

In many cases in the United Kingdom the application of reinforced concrete to quay and jetty construction has been brought about by economical necessity rather than by ideas of progressiveness *per se.* Most of the waters of the British ports carry teredo, and in many cases no timber but greenheart or jarrah will withstand its ravages. The Hennebique system of "ferro-concrete" construction, introduced into England by the late M. Mouchel, was first applied to sea-works at Southampton and, about the same time, to a jetty in the River Thames, at Purfleet. Both these works are referred to in detail in the paper. The success which attended them probably contributed very materially to the later popularity of reinforced concrete.

In general, under average British conditions, greenheart construction is about as expensive as reinforced concrete, pitch pine being, of course, considerably cheaper than either. Hence, in the absence of special conditions (such, for example, as those at Port Talbot mentioned by the author), pitch pine may be used in places where the marine borers are not too active, whereas in other locations either hardwood or reinforced concrete may be used, according to the local balance of general economy. In connection with this, the proved low maintenance cost of concrete work is a most important factor, and it is significant that in most cases where concrete pile-work has been introduced, its use has been continued.

The remarkable resilience of concrete piles under driving has been a most important factor in the increase in their use, but the writer thinks that the particulars given by the author on page 967* are scarcely applicable to a discussion on pile-work. It is reasonable to suppose that a hollow post of the dimensions given, reinforced with a large number of small rods, will be much more flexible than an ordinary pile, and such is the case. The resilience of piles is best

* *Proceedings*, Am. Soc. C. E , for April, 1914.

exemplified by the treatment that they are able to stand in ordinary use, and particularly in driving. In the early days, the interposition of a special dolly between the pile and the monkey was considered essential, but since it has been found that properly arranged spiral winding is able to withstand all the punishment that the pile-driver can give it, the dolly is no longer used. This, in itself, is quite sufficient proof of the resilience of reinforced concrete piles; and there is plenty of other evidence. Mr. Walker.

A point which has arisen and which has been much discussed more particularly in connection with marine work in England is the subject of the relative advantages of pre-moulded members and members formed *in situ*. It has been contended that the former method is the better one by reason of its being more convenient, but the writer believes that, except where the special conditions of the case preclude it, it is desirable to build the structure as monolithic as possible. No matter how carefully the work is carried out, it is hardly possible to unite satisfactorily hardened concrete several weeks old with the new concrete that must be put in to form the joint; nor can one get in built-up work the strength obtained by the continuity of the steel from one member to another in monolithic work. The use of additional binding steel at the joints does not fully compensate for this. Advocates of the built-up system put forward the claim that monolithic work may be considerably weakened by the wrong disposition of the reinforcing bars, and that, therefore, it requires considerable and constant supervision; but surely the increased relative importance of the field-work in the case of pre-moulded construction entails quite as much detailed supervision on the job.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

SOME PRINCIPLES RELATING TO THE ADMINISTRATION OF STREAMS.

Discussion.*

By Herbert E. Bellamy, Assoc. M. Am. Soc. C. E.

Mr. Bellamy.

Herbert E. Bellamy, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this paper with particular interest, not only on account of his familiarity with the subject, having dealt with questions of riparian rights in England, and of his knowledge of the systems in vogue throughout Australia, but because of the comprehensive and lucid manner in which the author has dealt with the whole subject. Indeed, the paper embraces practically every phase relating to the betterment in the administration of streams, so that there is very little, if anything at all, to add. The preamble presents amending principles suggested for adoption impartially, and with which, in every case, the writer cordially agrees. The paper brings together, in a small compass, a great mass of information dealing with the application of the doctrine of riparian rights in many countries, which cannot fail to be of the greatest value in the discussion of the subject by engineers.

Some years ago, when the writer was investigating some so-called riparian rights in connection with a proposed new water supply for a town in England, he was greatly puzzled to know what these rights were. Decisions of the Courts did not help him, because, as the author remarks, if the doctrine of riparian rights were carried into effect rigidly, water could not be diverted from a stream in any way. Several decisions could be quoted to support this latter contention if necessary, but the following brief abstract from a well-known case

*This discussion (of the paper by Clarence T. Johnston, M. Am. Soc. C. E., published in May, 1914, *Proceedings*, and presented at the meeting of September 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Bellamy. merely serves to show that something more tangible should be substituted for the absurd and diabolical doctrine, as it exists in many places to-day. In the case, Mason *v.* Hill, it was decided as follows:

"A riparian proprietor can have no larger right than he has by nature against those above and below him. Hence the right to have a stream to flow in its natural state without diminution or alteration is an incident to the property in the land through which it passes * * *."

The author points out that the first principle which should be recognized is that of public ownership of streams, lakes, and other bodies of surface water. The writer would also include "all underground supplies." This recognition is the fundamental principle of success of stream administration throughout the Australian States. In the writer's opinion, the second principle, *b,* which the author does not discuss at great length, is the positive safeguarding of the interests of the first user of the waters of a stream as against later comers, even including State or National rights.

As the author mentions that the modern system adopted by Australia merits study, the writer submits the following brief digest of the Water Rights Act of 1912 which is in force in the State of New South Wales.

The Water Rights Act came into force in 1896 and was consolidated in 1902. On November 26th, 1912, assent was given to an Act consolidating the separate Acts relating to Water Rights, Water and Drainage, Drainage Promotion, and Artesian Wells. This Act is administered by the Minister for Public Works. On the passing of the original Act, the right to the use and control of all streams, whether perennial or intermittent, flowing in a natural channel, through or past the land of two or more occupiers, and of all lakes, swamps, lagoons, or other sheets of still water, whether permanent or temporary, situate within or fronting the land of two or more occupiers, passed from the owners or occupiers of the said land and became vested in the Crown; subject, however, to the reservation that the said occupiers shall have the right to use the water on their frontage for domestic or stock purposes, or for gardens up to 5 acres in extent, without the necessity for obtaining a license for any work used solely for those purposes.

Rights granted under the Mining Acts on any public or private statute, are also preserved.

Section 8 provides for the rights of the Crown in respect of works, and Section 9 for the rights of the occupiers to which the Act applies.

The definition of "Work" is comprehensive, and includes any dam, lock, reservoir, weir, flume, race, channel (whether an artificial channel or a natural channel artificially improved), any cutting, tunnel, pipe, sewer, and any machinery and appliances; and "Work to which

Mr. Bellamy.

this part extends" means work connected with any river or lake flowing through, or past, or situate within the land of two or more occupiers, or with any water flowing in, to, or from, or being in any river or lake flowing or situate as aforesaid, whether such work be for water conservation, irrigation, water supply, or drainage, and whether such work be constructed before or after the commencement of this Act.

Section 10, and succeeding sections, provide for the granting of licenses to private individuals. Any person being in actual occupation of the site of a proposed work may make application for a license. The application should be forwarded to the Minister for Public Works, who will then advertise its receipt, together with a date and place at which a public inquiry will be held as to the advisability of granting the application. These inquiries are usually held by the local Land Board, and any person affected by the application may attend and give evidence for or against the granting of the application. Any person aggrieved by the finding of the Board may appeal to the Land Appeal Court within 28 days of the date of such finding. The finding of the Board is published in the *Government Gazette,* and, after the expiration of 30 days from the date of publication, the Minister, where the Board recommends the granting of the application, and provided no appeal is pending, shall issue a license for such period, not exceeding 10 years, as he may think fit, subject only to such conditions, if any, as may be embodied in the Board's finding. The applicant is notified by the Minister of the term for which it is intended to grant a license, and the amount of the fee which must be paid before a license can be issued.

Section 13 of the Act provides for joint application by two or more occupiers, as, for instance, persons on opposite sides of a stream who desire to construct a dam.

Section 17 is very important, because it secures to the holder of a license quiet enjoyment, and the sole and exclusive use, of the licensed work as against all persons, including the Crown.

Section 18 provides for a penalty for any alteration of work during currency of a license.

Sections 19 and 20 provide for the carrying out of works by the Crown, the formation of "districts" benefited by such works, the assessment of charges in each and every case for benefits accruing to the properties within the "district", and the payment of charges by the persons benefited. The actual procedure is conducted as follows:

"The Governor notifies by proclamation in the *Government Gazette* a proposal for the construction of a dam, lock, weir, channel or drainage work, as the case may be, to be constructed by the Crown, together with an estimate of cost. This notification is followed by the gazettal

Mr. Bellamy. of the boundaries of a 'district,' the land within which will, in the opinion of the Local Land Board, be benefited by the construction of the proposed work, and within which water or drainage charges may be levied. It is then necessary that a 'two-thirds majority' of the occupiers, who must also occupy an area exceeding two-thirds of the total area within the proclaimed district, petition the Land Board, on the proper form, to carry out the proposed work. In default of this petition the proposal lapses. On receipt of the petition, the Land Board may report to the Minister, recommending that the work be carried out, and after the expiration of thirty days from receipt of the Board's recommendation, the Minister may carry out the work with funds legally available. On completion of the work, the Land Board, by direction of the Minister, assesses the charges to be paid by each individual occupier, and the aggregate of these charges must not exceed six per cent. on the total cost of the work. The charges commence from the date of completion of the work, which must be advertised in the *Government Gazette*. The assessment takes place in open court, and the persons affected may attend and give evidence on their own behalf. On the petition of persons liable to pay one quarter of the total assessment, the Land Board must make a fresh assessment."

The remaining Sections, 21 to 27, refer to miscellaneous matters such as injuries to works; power of entry; obstructing persons in the performance of duties; recovery of fees, charges, and penalties; appeal; consolidated revenue; and the power to make regulations.

The regulations at present in force provide penalties for pollution, obstruction of or interference with the water in any stream or lake, and for causing trees, or débris of any kind to fall into any stream or lake, and also prescribes the various forms to be used under different sections of the Act.

From a perusal of these Acts,* it will be found that many of the principles advanced by the author have been provided for and are in successful operation in Australia.

The author certainly deserves the thanks of all water engineers for the exhaustive and thorough manner in which he has treated a most delicate and vexed subject.

* Copies of the Water Rights Acts at present in force in three of the Australian States, namely, Act No. 44, 1912, New South Wales; Act No. 2016, 1905, Victoria; and Act No. 25, 1910, Queensland; are on file in the Library of the Society.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE CONSTRUCTION OF THE KLONDIKE PIPE LINE.

Discussion.*

By Messrs. G. B. Pillsbury and Walter S. Wheeler.

G. B. Pillsbury, M. Am. Soc. C. E. (by letter).—The description in this paper of the frozen soil in the Klondike is most interesting, but should perhaps be supplemented in order to prevent a misconception of the difficulties and possibilities of earthwork in Alaska. The writer was engaged for several years in wagon-road construction throughout the occupied portion of the Territory, and acquired a fairly intimate acquaintance with its frozen ground. Mr. Pillsbury

In the interior of Alaska—the area north of the enclosing coastal range—the mean annual temperature is below the freezing point. As the temperature of the ground some feet below the surface must be approximately the mean of the surface temperature, it follows that this ground is, in general, permanently frozen. The underground drainage being thus sealed, the ground-water level is high, and the country is extremely swampy. Deposits of swamp muck, therefore, are very common in the valleys and elsewhere, and they sometimes contain lenses of solid ice several feet in thickness. As the author remarks, when the moss that covers this muck is stripped, its ice content melts under the summer sun. The writer has often seen a narrow ditch quickly enlarge into a wide and deep gully through this action. It is very obvious that if ice is built on, it must be kept frozen, and in order to keep it frozen, its protecting coat of moss must be retained.

* This discussion (of the paper by W. W. Edwards, Assoc. M. Am. Soc. C. E., published in May, 1914, *Proceedings*, and presented at the meeting of September 2d, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Pillsbury. It by no means follows, however, that all the valleys are covered with muck and ice, any more than all the valleys in more temperate zones are covered with muck and water. Ordinary earth lying below the ground-water, or rather ground-ice, is frozen throughout the summer, and the difficulties of working it, with the general sloppiness of the earthwork accomplished, are similar to those encountered in work in the very early spring in the northern part of the United States. It can be loosened, however, by blasting, and the difficulties in handling are merely those common to water-saturated earth. Earth above the ground-ice level thaws and dries readily, and its working is entirely normal.

The writer has traveled over a railroad near Nome to which the author possibly refers in the latter part of his paper. It was a very light surface road, and had the misfortune to be located, for the greater part of its length, on a frozen swamp. Light ditches had been dug through the moss and turf on each side, and these had gullied out so badly that much of the track had to be shifted to one side. The journey of about 60 miles occupied an entire day, although the passengers escaped the usual upset. The writer, however, does not think that any moral can be drawn from this railroad; a swamp does not afford the best location for a railroad in any country, and this road was merely fortunate enough to find its swamp frozen.

The prediction is ventured that good railroad construction in Alaska will follow standard practice closely, henceforth as well as heretofore. Swamps will be crossed on a fill, as usual. The principal departure from construction elsewhere will be in the matter of unit costs, and these, on account of the short working season and distance from supplies, will be several times greater than on any road in the United States.

Mr. Wheeler. Walter S. Wheeler, M. Am. Soc. C. E. (by letter).—Records at Skagway, Alaska, show that during the open season of 1907, the Guggenheims shipped into Dawson more than $5 000 000 worth of freight, a considerable portion of which was material for the Klondike pipe line of the Yukon Gold Company. The freight was landed, from ocean steamships, at the wharf in Skagway, and transhipped, over the White Pass and Yukon Narrow-Gauge Railroad, 111 miles to White Horse, and then by steamer down the Yukon to Dawson. The freight and passenger rates over this road are naturally quite high, as a considerable portion is built through exceedingly mountainous country, and the bulk of travel is confined to the summer, when navigation is open on the Yukon River. The track has a 3.9% grade for about 16 of the first 20 miles out of Skagway, to the famous White Pass, which is about 2 800 ft. above Skagway and mean tide. This portion of the road cost $2 000 000, or an average of $100 000 per

mile, and it is said that the road more than paid for itself during the first two years of operation. Mr. Wheeler.

Mixed trains leaving Skagway are drawn by four engines distributed equally throughout the same. At a point 8 miles from Skagway, a rock thrown into the river from the track struck the water in 5 sec.

By the "Far North", the writer assumes that the author includes all of Alaska and the adjoining Yukon Territory, and also British Columbia lying directly east of Alaska.

Costs for material, labor, etc., vary considerably for different locations. At the time the Klondike pipe line was constructed, with common labor at $4 per day and board, the Alaska Road Commission was paying $2.50 per day and board on the trail out from Haines Mission, with a working season of 5 months in each location. Machinists were paid $5 per day in Skagway, with extra for overtime, and $1 per hour in Dawson, Yukon Territory, and the adjoining creeks. Cord-wood, the principal item of fuel, cost $5.50 per cord delivered in Skagway, and $15 per cord in Fairbanks, Alaska. Board and room, the best obtainable, could be had in Skagway for $40 per month, while the same class of board and room cost $75 per month in St. Michael, Alaska, and Dawson, Yukon Territory, and $125 in Fairbanks.

The cost of electric light in Skagway was 10 cents per kw-hr., and in Fairbanks, 51 cents. Business telephones cost $5 per month in Skagway, and $20 in Dawson and Fairbanks.

These prices were current about 1907 and 1908, and though some of them have changed from time to time, the writer has given them in order to show that they vary considerably in different locations of the "Far North", and that estimates for construction in the "Interior" are, as a rule, much higher than along the Coast.

A fair grade of coal could be purchased in Skagway for $13 per ton, but, in St. Michael, the writer has paid $25 for coal of the same grade. This coal was all shipped from the United States or from lower British Columbia, while billions of tons of the best bituminous and anthracite are waiting to be mined along the coast of Southern Alaska as soon as the Government sees fit to allow it to be opened up.

At Skagway the temperature seldom falls below zero, but it is not unusual to experience a temperature of —70° Fahr. in midwinter at Dawson, Fairbanks, and Nome, and sometimes —80° Fahr. When the temperature falls below about —40° Fahr., there is never any wind, and, the air being very dry at that temperature, the real danger to life is from exposure when it appears to be warmer than it really is.

In summer, ordinary vegetables are grown in abundance, and wild raspberries, blueberries, cranberries, and currants are plentiful.

An excellent breeding place for mosquitoes is formed below the niggerheads (bunches of coarse wild grass) on the tundra, and during

Mr. Wheeler. May, June, and July, it is impossible to work on the tundras without the protection of nets and gloves; but August and September are comparatively free from these pests, which are not fever-bearing in the North.

It is impossible to make foundations to any great extent with the peat taken from the tundras, as it will dry out and burn up in the numerous tundra fires during the summer, as mentioned by the author in the case of the Nome Railroad.

In places near fresh and salt water, where suction dredges are available, good foundations can be made by running the discharge pipe so that the material will be deposited on the tundra, provided the water is not so rough as to prevent the dredge from operating. The writer had charge of dredging the St. Michael Canal during the summers of 1908 and 1909, and the material discharged from the pipe line would have made excellent foundations on the tundra, if they had been desired. When carrying on construction near the water, it is well to quarter the men on houseboats anchored well out from shore, so that, while not on duty, they can get some respite from mosquitoes. In this way one can obtain a more efficient working crew.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

CALIFORNIA PRACTICE IN HIGHWAY CONSTRUCTION.

Discussion.*

By Messrs. Allen Hoar and W. C. Hammatt.†

Allen Hoar, Jun. Am. Soc. C. E. (by letter).—In Mr. Hammatt's very interesting paper, he has brought out very clearly that in California there is an abundant supply of first-class road material of all kinds, well distributed over the entire State, and that, therefore, poor roads there are inexcusable. The author, however, makes several statements, which, though true, in a sense, must be misleading to those who are not thoroughly familiar with all the conditions. The writer is pleased to note that Mr. Hammatt has spoken so highly of water-bound macadam. This type of pavement when well-drained, laid on a properly prepared sub-grade, and carefully constructed, has still a great deal of usefulness. Mr. Hoar.

Under the heading, "Oiled Macadam," the author says, "A very small proportion of the total area of oiled macadam pavement constructed has been successful." He also says, "even under the best conditions, this pavement has little justification." Now, as a matter of fact, as there has been some success, there is certainly some justification for its use. In the introduction to the paper Mr. Hammatt states that it has been only in the last 5 or 6 years that any real road construction has been undertaken in California, but that a great deal of graveling and oiling had been done as a temporary improvement. Such is the case, the first practice of oiling being merely to afford

*This discussion (of the paper by W. C. Hammatt, M. Am. Soc. C. E., published in April, 1914, *Proceedings*, and presented at the meeting of May 20th, 1914), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Author's closure.

Mr. Hoar. a temporary relief from the dust nuisance and to protect earth and gravel roads from ravages by heavy rain storms. Observation of the results obtained by this treatment led to the adoption of this material in road construction, and finally to the present perfected type of bituminous macadam. In making so broad a statement, that only a very small proportion of the oiled macadam roads has been successful, the author has presumably taken stock of all those old oiled dirt roads, only a few of which can be considered at all as oil macadam.

It is true that at first there were no fixed specifications for work of this class. It was then in the experimental stage, and each engineer was doing his own experimenting at the expense of city, county, or State. In Los Angeles County alone probably $1 500 000 were practically wasted during this experimental stage, simply because the engineer then in charge refused to profit by the experience and advice of others, but would learn for himself. Other cases of the same kind have occurred, and these unfortunate conditions have done much to blacklist pavements of this type. At present engineers engaged in work of this class have adopted similar specifications and are obtaining general success.

The author states that the successful examples of pavements of this type have been either in localities where climatic conditions are such as to evaporate the more volatile parts of the oil, or where the application has been made in such a manner as to obtain the same results. Now, if success is obtained by a certain method in one place, would it not be good practice to adopt such a method in other localities? As a matter of fact, an asphaltic oil which will meet the requirements of the usual specifications as now adopted has practically no lubricating properties and very little volatile matter. The foundation stones are laid and rolled until a thorough mechanical locking of the particles takes place and it is as firm and solid as a water-bound macadam pavement; then the oil, at a high temperature, is sprayed on under pressure in such a manner that it is in a finely divided state and penetrates the interstices of the stone, acting primarily as a binder. Constructed in this manner, there can be no rolling or movement of the stones, as is pointed out by Mr. Hammatt. The wearing surface of finer aggregate is then laid on this foundation.

The patented form of Petrolithic pavement is by no means an oil macadam pavement. It is merely an earth road with sand and oil incorporated to a depth of 6 or 8 in., forming a thick rubbery mat, which is pushed and pulled by the traffic stresses until it is resolved into a series of humps and hollows. In a hot climate it remains soft and spongy, and soon becomes so wavy and covered with ruts that it is almost impassable. The Petrolithic Company's tamping roller, however, has come into very good use in compacting sub-grades and earth roads. The prongs, or sheep's feet, of this roller reach through

to the bottom of the sub-grade, and, starting there, compact it uniformly all the way to the surface, making a solid crust or arch to carry the weight of the finished pavement and its transmitted stresses. Mr. Hoar.

Probably the strongest argument against bituminous or oil-macadam pavements is the difficulty of obtaining good inspection, which must be had to secure satisfactory results. On the contrary, an argument put forth by the advocates of concrete roads is that, because of this lack of trustworthy inspection, concrete is the only type from which satisfactory and certain results can be obtained. In practice, however, this has proven to be far from the truth. The argument is that, no matter how little care has been taken in the preparation of the sub-grade, or how poorly the concrete has been mixed, it will become hard and form a good solid foundation, and will carry the traffic safely over the bad places in the sub-grade. Experience, however, has shown that, to secure satisfactory results with concrete roads, just as much attention must be given to the inspection as for any other type; for lack of the proper quantity of cement or poor and insufficient mixing is soon indicated by rapid disintegration.

The type of wearing surface which has met with the greatest success on roads having a concrete base consists of a $\frac{3}{8}$-in. coat of oil and crushed stone screenings ranging in size from $\frac{3}{4}$ to $\frac{1}{4}$ in. The oil is thus almost uniformly exposed to the action of the atmosphere, which causes it to "set", thus cementing the particles of the aggregate well together and to the base. As the coat is thin and uniform, it is easily applied, cheaply maintained, and permits of equal compression throughout the mass.

W. C. Hammatt, M. Am. Soc. C. E. (by letter).—As was expected, the writer's condemnation of oiled macadam pavements brought out some contradiction. Although this type of pavement has lost many of its advocates, there still remain many who believe in its value as a pavement. The writer, however, maintains that only a small proportion of the total mileage of oiled macadam pavement which has been laid, has been successful, and he does not include the oiled dirt and gravel roads under this heading. He knows of no road in the central part of the State, which has been under varied traffic for two years, which has remained in good condition without an expenditure of at least 8 cents per sq. yd. for repairs. There are some, where traffic is restricted to pleasure vehicles, which have remained in good condition, and these are cited as examples by advocates of this class of pavement. Mr. Hammatt.

The value of oil as a road material lies in the formation of a surface skin by the volatilization of the lighter constituents. This skin is very durable, and as long as it remains intact, it protects the general body of the pavement. In the southern part of the State, where Mr. Hoar's observations have largely been made, the sandy

Mr. Hammatt. nature of the soil and the hot, dry climate are such as to increase the thickness of the skin. Nevertheless, in examining the roads between Los Angeles and Pasadena, the writer has noted that those carrying the automobile traffic are the only ones which hold their shape.

To sum up, the justification for the adoption of a certain type of pavement can only be from its structural excellence or its economic value. One type will not be justifiable if another type will make a better appearance and keep in better condition at a smaller cost. The writer's experience has been that a protected water-bound macadam pavement will do this. This type may be made to fit all conditions by varying the thickness of the base and of the protective covering. A macadam pavement, 6 in. thick, with a skin protection of asphalt and screenings, will cost about 70 cents per sq. yd., and will require an annual expenditure of about 9 cents per sq. yd. for upkeep. Increasing the protective covering to 2 in. in thickness of asphaltic macadam increases the cost to about $1.10 per sq. yd., and the upkeep will be nil for from 10 to 15 years. Compare these costs, as well as the appearance and usefulness of the pavement, with oiled macadam under the best conditions, and the writer believes that there will be no justification for the latter.

A skin of oil and screenings has been applied to some of the concrete bases of the State highway for their protection. This has been quite successful, as pointed out by Mr. Hoar, for the reason that the oil does not penetrate the concrete, and the concrete, by absorbing heat, promptly carbonizes the oil and hardens the skin. Moreover, for so thin a coating, it is probable that oil is better than asphalt, as the coating will be more resilient and cause less impact on the concrete.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EMIL GERBER, M. Am. Soc. C. E.*

DIED APRIL 16TH, 1914.

Emil Gerber was born in Reichenbach, Saxony, on January 31st, 1858, and died in Pittsburgh, Pa., on April 16th, 1914. His father, C. F. Gerber, was born in 1819 and died in 1899. He was a designer and manufacturer of textile fabrics, and introduced the use of power looms in his native town. His mother's maiden name was Christliebe Klotz, daughter of Carl Klotz. His ancestors for several hundred years were residents and prominent citizens of Reichenbach, Saxony. His father came to the United States in 1862 and settled in Webster, Mass., and in 1867 his mother came to Webster, bringing Emil and his two brothers, Herman and Carl.

He was educated in the common schools of Webster, Mass., and the Stevens High School at Claremont, N. H. In the fall of 1873 he entered the Worcester Polytechnic Institute, at Worcester, Mass., and was graduated as a civil engineer in the spring of 1876, being at that time only a little more than eighteen years of age. A classmate states that he was very much liked by all the students and professors. He was studious, a man of high character, and at that time gave promise to do good work. The President of the Institute writes: "Few students have done better in the Institute."

Mr. Gerber taught school for one year at Southbridge, Mass., and was employed as a bookkeeper in a corporation store connected with the woolen mills in Webster, Mass., until May 1st, 1879, when he was engaged as a Transitman on the Fremont, Elkhorn, and Missouri Valley Railroad. In May, 1880, he became Locating Engineer with the same railroad, and in 1881 was promoted to the position of Assistant to the Chief Engineer, Captain James Edward Ainsworth. On August 1st, 1881, he was appointed Assistant Engineer on the Blair Bridge over the Missouri River, near Blair, Nebr., built by the Sioux City and Pacific Railroad which later became a part of the Chicago and Northwestern Railway. The late George S. Morison, Past-President, Am. Soc. C. E., was Chief Engineer of this bridge. Toward the completion of this work, on November 1st, 1883, Mr. Gerber was made Resident Engineer.

In October, 1884, he returned to his former position with the Fremont, Elkhorn, and Missouri Valley Railroad, but still retained charge of the Blair Bridge and its extensive protection works.

* Memoir prepared by Otis E. Hovey and August Ziesing, Members, Am. Soc. C. E.

In November, 1885, he was appointed Assistant Chief Engineer of the same railroad, which position he held until he resigned on June 15th, 1887, to become Resident Engineer of the Sioux City Bridge over the Missouri River, of which Mr. Morison was also Chief Engineer. This bridge was completed at the end of 1888, and Mr. Gerber was then appointed Resident Engineer of the bridge over the St. Johns River at Jacksonville, Fla., which was being built by the Jacksonville, St. Augustine, and Halifax River Railroad, with Mr. Morison as Chief Engineer. A severe illness prevented Mr. Gerber from completing this bridge, and in May, 1889, Mr. Morison placed him in charge of his Chicago office, where he remained until the fall of 1897.

During this period, he was connected with all Mr. Morison's important works, among which may be mentioned the Cairo, Memphis, Burlington, Winona, Bellefontaine, Alton, and Leavenworth Bridges; the Chicago, Burlington, and Quincy entrance into St. Louis, and many less important works.

In 1897, Mr. Gerber resigned his position with Mr. Morison to become Chief Engineer of the Lassig Bridge and Iron Works, in Chicago, Ill. He held this position until the spring of 1900. When the American Bridge Company was formed, he was made Manager of the Lassig Plant. In July, 1901, he was promoted, becoming Assistant to the President of the American Bridge Company, in Philadelphia, Pa. Early in 1904, he went to Pittsburgh, in the same position, and in the fall of 1905 was made Operating Manager of the Pittsburgh Division in addition to his position as Assistant to the President. In 1911, he was relieved as Operating Manager and made General Manager of Erection, but still held the title of Assistant to the President. He held this position during the remainder of his life.

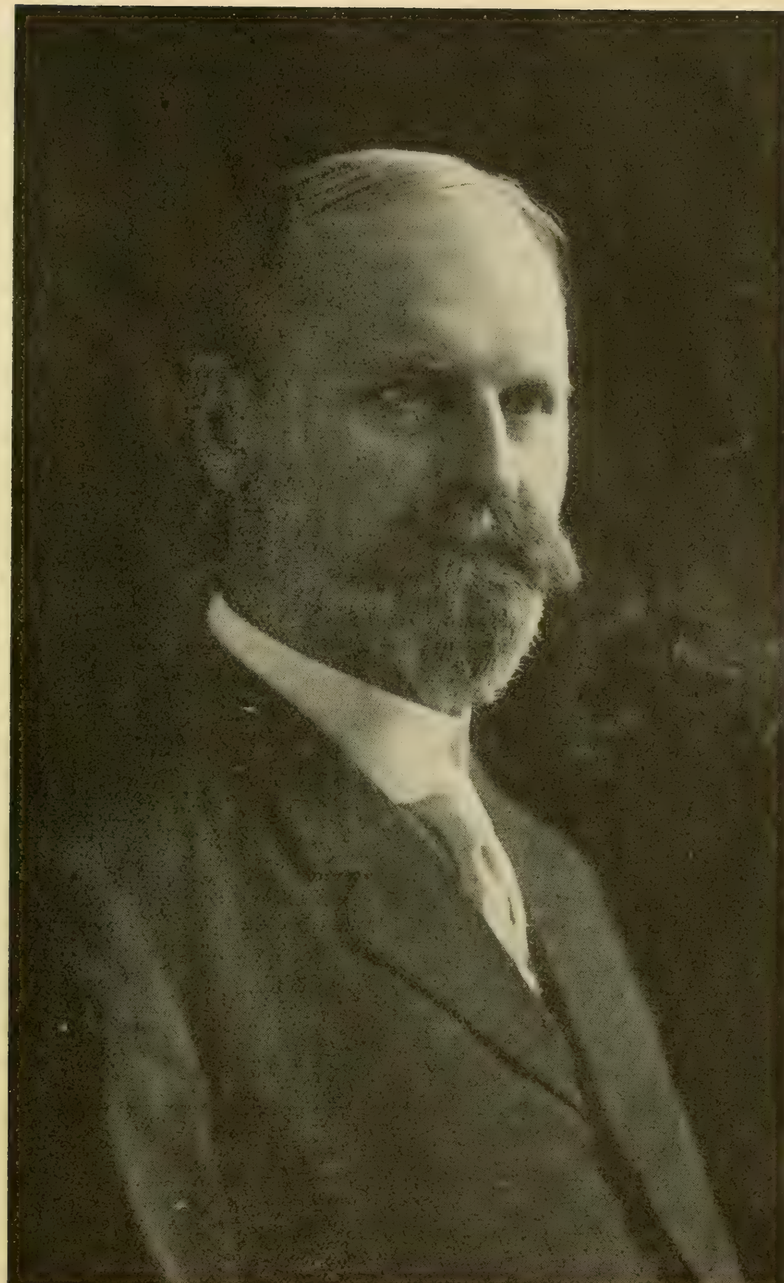
He was married on January 3d, 1882, at West Roxbury, Mass., to Caroline Herthel, daughter of F. J. Herthel, Sr., and is survived by his widow, a daughter, Mrs. Laura E. Olson, of Chicago, Ill., and a son, Emil Gerber, Jr., of Pittsburgh, Pa.

As is often the case with engineers fully and continuously engaged on important work, Mr. Gerber was not a frequent contributor to engineering literature, but whatever he wrote is strongly marked by his vigorous personality.

He was the author of a paper entitled "Painting of Iron Structures Exposed to Weather",* and also of one entitled "Some Commercial Features of Structural Engineering".† He had based a few lectures to young engineers on the latter paper, and his last brief illness and death prevented him from lecturing on the same subject before the American Bridge Engineering Club of New York. This paper is one which should be read by all engineers.

* *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 485.

† *Proceedings*, Eng. Soc. of Western Pennsylvania, Vol. XXIII, p. 125.



E. Gerber

It frequently happens that an engineer employed by individuals or corporations does not become so well and favorably known as he would have been had he been in practice under his own name. It is also common for an engineer associated with corporations to specialize along some narrow line of engineering work. Mr. Gerber was an exception in these respects. His early experience covered a very wide range of work, including railroads, difficult foundations, construction of bridges and other structures pertaining to railroads, and, at the time he took up his work as an engineer associated with a corporation, his experience was wide and his judgment well trained.

Those closely associated with him were impressed by the thorough manner in which he carried out any work on which he was engaged, his genial and helpful disposition, and his hearty and liberal endeavors to assist those around him in every way. He was a man who would ungrudgingly draw from his experience anything that would help his associates to grasp an engineering or business situation with which he was familiar.

He never did anything superficially. Any problem that was before him received his complete and earnest attention. Whenever he undertook an investigation or made a report, one could depend on his having gone into the matter thoroughly and having expressed his best opinion; and any facts contributed to the subject could be depended upon.

Mr. Gerber had a strong mentality and firm opinions on any subject to which he had given thought, but at the same time was always willing to discuss exhaustively the opinions of others, and when they were backed by facts and good reasoning was ready to modify his previous views in an eminently fair-minded manner.

A notable trait of his personality was his unusual ability in the establishment of cordial relations with his associates, and his interest in all that contributed to their welfare.

On account of his connection with a multitude of engineering works to which he contributed in more or less degree, and in which his strong common sense and clear thinking determined important decisions concerning designs and methods of construction, it is unfortunate that the Profession cannot point definitely to this or that work as purely his own. His monument is a more personal one. His strong, honest, thorough, and genial personality has impressed itself on all who knew him, and most of all on those who knew him intimately. Such characters are an inspiration to all members of the Profession.

Mr. Gerber was elected a Member of the American Society of Civil Engineers, on February 1st, 1888, and served as a Director from January, 1912, until the time of his death. He was also a member of the American Railway Engineering Association, the Western Society of Engineers (Past Treasurer), the Engineers' Society of Western Pennsylvania, the Chicago Engineers Club, and the Duquesne Club and the Junta Club, both of Pittsburgh.

HENRY FRANCIS LABELLE, M. Am. Soc. C. E.*

Died December 12th, 1913.

Henry Francis Labelle was born in Montreal, Que., Canada, on January 31st, 1860. His father, Regis Labelle, was a lumber merchant, and was a native of Jirome, in the Province of Quebec. His mother, Lucille Marceau, was born at Lacolle, in the same Province.

After his graduation from St. Mary's College, Montreal, he entered the Montreal Polytechnic School, from which, in 1882, he received the degree of Civil Engineer.

Mr. Labelle's first employment was as Rodman and Draftsman for the Chicago, Burlington and Quincy Railroad. In August, 1883, he returned to Canada, and, after about a year in the Dominion Lands Office, entered the office of the Chief Architect, Department of Public Works, Canada, where he was employed until 1890, during the last two years as Assistant Superintendent of Public Buildings. From 1890 to 1895, he was employed as Assistant Engineer or Engineer in Charge of the design and construction of numerous municipal water supply systems in New York, Pennsylvania, and Maryland.

By disposition and training Mr. Labelle was peculiarly fitted for the varied and interesting problems of hydraulic and hydro-electric engineering design, and after once entering this branch of the Profession, he did not leave it.

He was for several years in the employ of the East Jersey Water Company, the latter part of the time as First Assistant Engineer in charge of the Montclair office and of construction work amounting to approximately $1 000 000. In 1895, and again in 1900, he was employed by the Susquehanna Power and Paper Company, on surveys and designs for a 35 000-h.p. development on the lower Susquehanna River, and in 1902 by other parties on investigations and studies of power possibilities on the same river at Turkey Hill.

In 1901, he was appointed Designing Engineer on a new water supply for Santiago, Cuba, for the Military Government. In the latter part of 1902, he returned to Cuba where, for some time, he was engaged by Havana interests as Engineer to report on or design water supply and electric power and light systems.

In 1904 Mr. Labelle accepted employment under the Philippine Government, first as Engineer in Charge of all irrigation work in the Islands, under the Director of Public Works, and later as Principal Assistant Engineer on the construction of the new Manila Water-Works. While in charge of the irrigation works, he prepared a system of irrigation laws for the Islands.

In 1907, on account of threatened lung trouble, he was advised by his physicians to leave the Philippines. He returned to the United

* Memoir prepared by John T. Whistler, M. Am. Soc. C. E.

States, and located in Silver City, N. Mex., where he remained until 1911, when heart trouble compelled him to change from the high altitude to a lower one, and he moved to Albuquerque, N. Mex., where he remained until his death.

It was typical of his character that he wrote of his heart trouble in 1911, "I now have the choice of two ways to die. I can either remain in Silver City and die of heart trouble, or I can go to Albuquerque and die of lung trouble."

After several months of correspondence and inquiry as to his experience and qualifications, he was notified, on November 5th, 1911, of his employment by the Argentine Government to act as Adviser in the development of an Irrigation Bureau. It was just at this time that his complication of heart trouble had developed, and, much to his regret, he was compelled to decline the office.

In 1908, he was married to Miss Emily Rose Woodford, of London, England, who for several years had been a resident of Philadelphia, Pa., and who survives him.

Mr. Labelle was the author of several discussions published in the *Transactions*, and had also contributed to various current technical journals. Though of foreign birth, he took out naturalization papers and became a citizen of the United States in January, 1904.

Altogether, Mr. Labelle was one of the sweetest and most noble characters with whom it was one's privilege to associate. He never complained, rarely spoke ill of any one, and then only with that calm judgment which guided his whole career. He never was known to do any one the slightest injustice. Simply to have known him was a privilege, to have known him intimately was an inspiration.

Mr. Labelle was elected a Member of the American Society of Civil Engineers on April 6th, 1898.

FRANK PARSONS LANT, M. Am. Soc. C. E.*

Died June 30th, 1914.

Frank Parsons Lant was born at Nassau, N. Y., on May 24th, 1858, and was of Colonial Dutch descent.

His business career, which included many important positions, was begun on September 26th, 1881, when he was employed as Axeman, and, subsequently, as Rodman and Assistant Engineer on the location and construction of the West Shore Railway, holding the latter position until March 1st, 1884.

From June 1st, 1884, to December 31st, 1886, Mr. Lant was City Engineer and Surveyor of Hudson, N. Y., designing and constructing

* Extract from memoir prepared by Gerald E. Hart, Esq., of New York City and Jacksonville, Fla., supplemented by information on file at the Society House.

sewers, highways, etc. He was also engaged in the private practice of surveying and engineering during that time.

During 1887, and until March 1st, 1888, he was employed on the location of a branch line for the Erie Railway, in Pennsylvania, and also made the surveys and had charge of the construction of the foundations for the Myrtle Avenue Elevated Railroad, in Brooklyn, N. Y. During this period he made the preliminary and location surveys for the New York and Massachusetts Railroad, from Ancram Lead Mines, N. Y., to near Westfield, Mass.

From March to November, 1888, he served as Division Engineer on the preliminary and location work for a line over the Blue Ridge Mountains, north of Marion, N. C., for the Charleston, Cincinnati and Chicago Railway, and from November, 1888, to March, 1889, he was engaged on a revision of the location of the New York and Massachusetts Railroad from Ancram Lead Mines, N. Y., to Chicopee Falls, Mass. In April, 1889, he was placed in charge of the reballasting of 60 or 70 miles of roadbed for the Rome, Watertown and Ogdensburg Railroad.

In November, 1889, Mr. Lant went to Jamaica, West Indies, where he remained until July, 1891, making location surveys for the West India Improvement Company.

From August, 1891, to February, 1896, he was engaged in the private practice of engineering and surveying, during which time he made the plans for the disposal of ashes in the filling in around Riker's Island, a survey for a railway proposition on Long Island, etc., etc.

In connection with the public works of New York City, Mr. Lant served the City Government, as follows: From February, 1896, to October, 1897, he was Engineer Inspector in the Department of Public Works; from October, 1897, to July, 1898, Engineer of Grades (Commissioners' Engineer) on the electrification of the Second Avenue Railroad; from August, 1898, to February, 1902, Transitman and Computer in the Topographical Bureau of the Board of Public Improvements; and from February, 1902, to March, 1907, Transitman and Computer in the office of the Engineer of Street Openings.

In March, 1907, he was appointed Manager (which title also included that of Chief Engineer and Surveyor) of the Lawyers' Engineering and Surveying Company, of New York City, which position he held at the time of his sudden death.

Mr. Lant was married on November 11th, 1891, to Stella, daughter of the late Jacob L. Seixas, who, with his father and mother, survives him.

He was an ardent student in his chosen profession, to which he did honor. His capabilities were such that his services and advice were frequently sought in cases of special importance. He possessed

in a high degree the power to command loyal service from his subordinates, as well as the confidence and respect of all who knew him. His genial manner and kindly nature will long be remembered by many who, in his death, experience a sense of personal loss.

Mr. Lant was elected a Junior of the American Society of Civil Engineers on October 3d, 1888, and a Member on February 1st, 1910.

ULYSSES STANISLAUS LUTZ, M. Am. Soc. C. E.*

Died December 8th, 1913.

Ulysses Stanislaus Lutz, the son of Ignatius and Rose Jacwuin Lutz, was born in Philadelphia, Pa., on May 25th, 1853. He received his earlier education at Roth's Military Academy, and was graduated in 1873 at the head of his class from the Philadelphia Polytechnic Institute, with the degree of Civil Engineer.

In 1873 he was engaged as Rodman on the South West Pennsylvania and the Lewisburg, Centre, and Spruce Creek Railroads. In 1874 he served as Leveler on the Tyrone and Clearfield and New River Railroads; in 1875 as Rodman on the Pennsylvania Railroad; in 1876, as Transitman on retracing the old Portage Railroad; from 1877 to 1879, as Assistant Supervisor, Maintenance of Way, on the Pittsburgh and Philadelphia Division, of the Pennsylvania Railroad; in 1880, as Supervisor, Maintenance of Way, on the Shenandoah Valley Railroad; from 1881 to 1885, as Assistant Engineer on Surveys for the South Penn Railroad, for two years, and for two years as Resident Engineer in charge of Subdivision B, Division 5, of the South Penn Railroad, in Somerset County; in 1886, as Principal Assistant Engineer on the location and construction of the Bloomsburg and Sullivan Railroad; in 1887, as Principal Assistant Engineer on the location of the Baltimore and Drum Point Railroad; in 1888 as Construction Engineer of the 25th Ward Gas Works, in Philadelphia, Pa.; in 1889, as Division Engineer on the location and construction of the Cumberland Valley Extension of the Louisville and Nashville Railroad.

For a time Mr. Lutz followed private practice in Virginia and North Carolina, and, in 1892, became Constructing Engineer for the Pennsylvania Iron Works. In 1893 and 1894, he was Chief Engineer on the location and construction of the South Jersey Railroad, and thereafter was engaged on the construction of electric railroads in and around Baltimore, Md., having been, in 1895 and 1896, Manager of Construction for the Baltimore and Catonsville Railroad.

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* Memoir prepared by Charles W. Staniford and S. L. F. Deyo, Members, Am. Soc. C. E., and John H. Frazee, Assoc. M. Am. Soc. C. E.

In 1897, he entered the service of the City of New York in the Department of Public Works, being first engaged on the Park Avenue Change of Grade Improvement from 56th to 96th Streets in Manhattan.

Mr. Lutz left the City's service to go with his former Chief, the late William F. Shunk, as Chief Assistant Engineer on the location and construction of the Quito and Guayaquil Railroad, in Venezuela, and again to go with the Iowa and Minnesota Railroad, as Assistant Engineer on Construction.

In 1905, he entered the Department of Finance of the City of New York, as an Assistant Engineer, and held that position until his death, which occurred on December 8th, 1913, after an illness of three weeks.

Mr. Lutz's practical knowledge acquired through a long and varied engineering experience was of great service to the City. He was an indefatigable and conscientious worker, and those with whom he was associated were impressed with his sense of loyalty to the City and to his friends.

Mr. Lutz was elected a Member of the American Society of Civil Engineers on March 5th, 1912. He was also a member of the Masonic Fraternity.

LEON LINCOLN GAY, Assoc. M. Am. Soc. C. E.*

Died May 6th, 1914.

Leon Lincoln Gay, the only son of Carlos E. and Calista Gay, was born at Barton Landing, Vt., on May 30th, 1879. He obtained his education at the public schools, and was a graduate of St. Johnsbury Academy. He then entered the Sheffield Scientific School, of Yale University, from which he was graduated in 1901, with the degree of Ph.B. While at Yale, he was a member of the Track Team and made some good records as a long-distance runner.

In the fall of 1901, Mr. Gay went to Idaho to become Assistant Engineer in charge of the construction of a small power plant at Horseshoe Bend, under A. J. Wiley, M. Am. Soc. C. E., Consulting Engineer. On the completion of this plant, he entered the United States Reclamation Service as Engineering Aid, and was engaged on the preliminary work for the Minidoka Project, in Idaho, during the greater part of 1903 and 1904. During the construction of the Minidoka Dam, on Snake River, he was Assistant Engineer on that work. On its completion, he was assigned to take charge of the building of the original Jackson Lake Dam, near Yellowstone Park, in Wyoming, and made an enviable record in completing this temporary

*Memoir prepared by C. W. Joslyn, Esq., and F. C. Horn, M. Am. Soc. C. E.

dam at a remarkably low cost. Later, he was Principal Assistant Engineer in charge of the construction of the Boise River Dam, of the Reclamation Service, near Boise, Idaho.

In 1908, Mr. Gay resigned from the Reclamation Service to accept a position as Engineer for a large irrigation enterprise in the Sacramento Valley, California. The following year he was attacked with tuberculosis, and the last five years of his life was a continuous fight with this dread disease to which he finally succumbed while undergoing an operation in Montreal, Que., Canada.

Mr. Gay was known by the large number of engineers of his acquaintance in the West to be a man of high moral character and absolute integrity. He was considered one of the most capable young engineers in the line of irrigation in the United States, and previous to his unfortunate illness he had executed some important and successful work. His misfortune and death are peculiarly sad. He was a typical New Englander, and among his near friends, his humor and originality gained for him the affectionate nickname of "the Yank".

He is survived by his father and stepmother, of Orleans, Vt., as well as by his two sisters, Mrs. Wallace C. Gilpin, of Barton, Vt., and Mrs. Fred E. Parker, of Falls Church, Va.

Mr. Gay was elected a Junior of the American Society of Civil Engineers on December 5th, 1905, and an Associate Member on October 2d, 1907.

PHILIP MORRIS PRITCHARD, Assoc. M. Am. Soc. C. E.*

DIED JULY 8TH, 1914.

Philip Morris Pritchard was born on April 29th, 1872, at Rock Ferry, near Birkenhead, England. He was educated in London, where his father was in business as a tea importer and merchant. At an early age the boy showed a strong liking for Engineering, and after his graduation from Kings College, London, in 1891, he was sent to Newcastle-on-Tyne to serve an apprenticeship of four years with the firm of R. and W. Hawthorn, Leslie and Company, Limited, Locomotive and Marine Engineers and Shipbuilders, where he received a thorough practical workshop training. During three years of his apprenticeship, he attended the Engineering Course at Durham College of Science, at Newcastle, and, on his graduation, was awarded an Associateship in Science.

In June, 1895, Mr. Pritchard was engaged as Draftsman and Assistant Engineer at the Tennant-on-Tyne Works of the United

* Memoir prepared by the Secretary from information supplied by the United Alkali Company, Limited, Widnes, England, supplemented by information on file at the Society House.

Alkali Company, Limited. In January, 1896, the Company organized its Engineering Department at Widnes, England, and he was brought from the Tennant Works to occupy an important and responsible position on the engineering staff at the latter place. This promotion eventually led to his appointment as Chief Engineer of the Company, a position which he held for about 12 years and at the time of his death on July 8th, 1914.

While in charge of the Engineering Department of the United Alkali Company, Limited, Mr. Pritchard was engaged in the design and erection of much new plant in the English and American works of the Company and did valuable work in its Spanish mines.

All through his career, Mr. Pritchard showed brilliant ability as an Engineer, and his death is a great loss to the Company as he was greatly esteemed both by the Directors and his colleagues.

He was a Member of the Institute of Mechanical Engineers and of the Society of Chemical Industry. He was also an Associate Member of the Institution of Civil Engineers.

Mr. Pritchard was elected an Associate Member of the American Society of Civil Engineers, on June 6th, 1900.

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"THE LOCK 12 DEVELOPMENT OF THE ALABAMA POWER COMPANY, COOSA RIVER, ALABAMA." E. L. SAYERS and A. C. POLK. (To be presented Nov. 4th, 1914.)

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